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Bridge Loops and Passenger Traffic Between Manhattan and Brooklyn.

One of the most complicated and troublesome questions to come before the Public Service Commission of New York is that of the passenger traffic between the boroughs of Manhattan and Brooklyn, and its bearing in some respects upon the increase of elevated railroad facilities in the former borough along the existing lines and the proposed loop between the Manhattan terminals of the Brooklyn and Williamsburgh bridges. The disgraceful and uncivilized daily crush at the Manhattan end of the Brooklyn bridge and the much discussed increased capacity of that terminal and of the bridge itself have already been subjects of the most vigorous and sometimes even fantastic consideration for a number of years. Several commissions and numerous self-appointed agents for the best solution of this important problem have put on record their conclusions, and the city has spent a considerable sum in testing at least one project. The last commission appointed by the Commissioner of Bridges reported at the end of 1906 a comprehensive plan, which has at least the merit of ultimate relief as complete as any that can be hoped for, and temporary relief to an extent limited only by the present train and structural capacity of the Brooklyn bridge.

It is altogether probable that whatever measures of relief may be perfected, it will scarcely be possible to install them before the demands of the situation will exceed their capacity. Nevertheless the present conditions are absolutely intolerable and in seeking for effective means for their cure it is essential that the broadest possible views of all elements of the problem shall be taken. There have been laid down with much persistence and emphasis by interested parties certain assumed general principles to control the solution of these difficult problems and to fix the conditions which may either govern them or result from them. The Public Service Commission will do well to recognize the fundamental differences which may exist between plans or propositions loudly advocated, frequently by representatives of semi-civic bodies, of one idea only, and those calculated to lead to the real relief of the public. What is imperatively demanded for the situation is an effective means for transporting people quickly and comfortably between the boroughs of Manhattan and Brooklyn so far as the East River bridges and the connecting lines are concerned and not the enforcement of some thesis strenuously enunciated in behalf of some impracticable proposition.

One of the most emphatic statements made in connection with the solution of this problem in its relation to the East River bridges is that there must be no addition whatever to the present elevated railroad facilities of the borough of Manhattan and consequently that there must be no elevated loop constructed between the Manhattan terminals of the Brooklyn and Williamsburgh bridges. It may be perfectly proper to assume the general principle that there should be no more elevated railroads constructed on Manhattan Island. The Engineering Record is inclined to take that view of the matter, and public sentiment probably strongly favors it; at the same time that is a long way from ruling out the construction of additional express tracks under proper conditions on the Second and Third Avenue elevated lines or even the construction of an elevated loop through streets well adapted to that purpose between the two bridges named. Elevated railways in the city of New York have given a measure of relief to its people impossible to have been secured in any other way, and they are there to stay. They will continue to render that service for an indefinite period in the future. It would be irrational and thoroughly absurd to take the position that while they have rendered, and will continue to render, such service they shall not be permitted so to develop their capacity as to greatly increase that service to the comfort and convenience of the community.

The Subway system of rapid transit certainly has extraordinary advantages, but it does not possess them all. An elevated loop between the Williamsburgh and Brooklyn bridges on the Manhattan side can be built much more quickly than a subway and when it is borne in mind that the Brooklyn bridge must be reconstructed with far greater capacity than at present, as soon as diversion of traffic to the Manhattan bridge will permit, it becomes highly probable that both subway and elevated loops will be required for the enormously increased traffic of the future, especially as both terminals afford ample facilities for the two classes of connections. It is petulantly idle and hypercritical to object that such loop connections give relief only to the rapid transit companies in handling their cars. That is precisely the kind of relief which they ought to have, for it constitutes one of the most important elements in efficient transportation. The masses of people whose comfort and convenience depend upon the best possible means of interborough rapid transit cannot be more effectively served than by such terminal and connecting facilities between the East River bridges and the adjacent transit systems, as will afford those systems the highest attainable efficiency in handling their cars. This is the very pith and essence of this great problem of rapid and comfortable transit to which the Public Service Commission must give its best thought and not be governed by the academic advocates of any one idea.

Accident Records in Equipment Maintenance.

The maintenance of engineering equipment includes some of the most interesting problems in the field of industrial technology. The reduction of repair costs to the lowest amounts consistent with safe and reliable service is always before the maintenance department as a desirable condition to meet, and the invention of labor and time-saving devices and methods always contributes towards this end if the quality of the work is not sacrificed in the meanwhile. As equipment is developed from year to year to satisfy varying specifications, it is always of importance to measure the effect of changes in design upon the total cost and facility of repairs and renewals, and in this connection the temporary or permanent failures of the machinery in service deserve careful and detailed study. Gradual deterioration of wearing parts is easily followed in most cases, for modern equipment is designed with accessibility as one of the important requirements to be met; but the study of sudden failures or accidents is much more difficult and often neglected by operating companies in comparison with the attention paid to current repairs and their contributory causes.

Serious accidents involving legal complications are generally analyzed at considerable length by executive and technical officers of industrial organizations, but even this work is apt to be carried out in a haphazard manner if the principal energies of the staff are occupied in replacing damaged equipment as quickly as possible in order to establish the normal rate of production again. Obviously, the return to normal conditions is usually the most important task after personal injuries are cared for, supposing that these appear in an accident, but it is well not to overlook the value of notes upon the conditions taken as soon as possible after the equipment goes wrong. In the operation of a modern street railway there is a wise appreciation on the part of the management of securing the fullest particulars of the smallest and most insignificant accidents as well as those of undisputed importance. Of course, the avoidance of damages through unjust claims on the part of the public is the main reason why street railway employees are, on large systems, provided with blank forms on which the minute details of each casualty are to be recorded, but apart from their legal aspect, such reports are often helpful in throwing light upon the performance of the equipment and suggestive of changes in design to avoid similar accidents in the future. In an industrial or power plant the use of blanks bearing upon probable accidents is seldom feasible to anything like the extent which obtains in street railway service, for in the latter work the range of accidents is limited to the car and its vicinity and certain general forms can be applied to practically every case. At the same time, reports on accidents to engineering equiment in general by technical employees to their executives can be made valuable analyses of weak spots in the apparatus if care is taken to make them up on the strength of full notes of the condition before, during and after the trouble occurs. Literary excellence in such reports is not the point—what is needed is a full memorandum of the conditions mentally noted, but seldom recorded in the haste of setting things to rights.

In the stress of emergencies there is often no time to make the notes of conditions, and no engineer with heavy responsibility can be expected to sit down and prepare a report on either major or minor accidents until all that is possible has been done to restore the original rate of production. Too often, however, the necessary notes are not made until some of the details at first clear have become uncertain. The value of all emergency notes lies in their accuracy in detail, however conflicting the data may appear at first. The smaller accidents are likely to be as suggestive of desirable improvements in equipment as casualties costing considerable sums to repair and offset.

The Civic Federation Investigation of Municipal Ownership.

The abstracts of reports made by members of the committee of the National Civic Federation appointed to examine public and private management of public service undertakings in this country and abroad, printed in this journal last week, must have seemed to some readers as partaking of the nature of a technical imitation of Joe Miller's joke book. An examination of the same data has led one set of critics to declare that municipal ownership is a failure and the other set to assert equally strongly that it is a complete success. It is hardly necessary to say that those who found muncipal ownership to be a success were well-known advocates of it, while the advocates of private ownership were gentlemen engaged in the management of public service corporations. Each party doubtless felt called upon to stick to the source of its bread and butter, even in cases of reasonable doubt. It seems a pity, however, that they could not agree somewhat, and thus make their statements carry some weight with the public, which must look upon the critical reports given out by the Federation's secretary as mere ex-parte statements, to be regarded merely as utterances of interested parties. The entirely divergent opinions which were stated in the abstracts printed last week show how futile it is to expect that those who are out-and-out advocates of municipal or private ownership under all conditions may ever reach conclusions at all harmonious.

The costly investigations conducted by the Federation must not be judged, however, by the statements of the partisan reviewers of the reports which were outlined last week. The main committee has been able to reach a number of highly important conclusions from the information collected by its investigators, and its report, signed by all members of the committee save Mr. Walton Clark, is of more than usual value and a contribution of particular interest at the present time. The committee which signs the report is made up of representatives of all classes of professions and trades. It represents all political parties and every shade of belief in municipal ownership or antagonism to it. Accordingly the practical unanimity with which certain important conclusions were reached in a gratifying indication that the work which has been carried on may be of some definite help in reconciling the present discordant views held on the subject. The conclusions reached are as follows: First, public utilities, whether in public or private hands, are best conducted under a system of legalized and regulated monopoly; second, public utilities in which the sanitary motive largely enters should be operated by the public; third, the success of municipal operation of public utilities depends upon the existence in the city of a high capacity for municipal government; fourth, the franchise grants to private corporations should be terminable after a fixed period, and meanwhile subject to purchase at a fair value; fifth, municipalities should have the power to enter the field of municipal ownership upon popular vote under reasonable regulation; sixth, private companies operating public utilities should be subject to public regulation and examination under a system of uniform records and accounts and of full publicity. Messrs. Chas. L. Edgar, of the Boston Edison Electric & Illuminating Co., and Mr. Walton Clark, of the United Gas Improvement Co., refused to concur in the second and fifth conclusions, and the reasoning upon which they are based.

For the successful municipal operation of a public service undertaking, the committee's investigations indicate, according to a report given out this week, that provision must be made for compliance with the following conditions: First, an executive manager with full responsibility, holding his position during good behavior, must be in charge; second, political influence and personal favoritism must be excluded from the management of the undertaking; third, the finances of the undertaking must be separate from those of the rest of the city; fourth, the necessary bond issue for revenue-producing utilities must be exempt from the debt limit law, and should be a first charge upon the property and revenues of the undertaking. These conclusions go practically to the root of the whole subject of the successful municipal ownership. At the present time there is little attraction to a man of even third and fourth rate calibre as a technical executive to enter municipal service. Except in a few cities where municipal affairs have been administered for many years on a business-like basis, the executive head of any department is likely to be chosen for political rather than business reasons. The fact that he often makes a good executive is due to the fact that, to a certain extent, political skill calls for much the same kind of ability that business success does. Wherever technical matters are involved, however, as they always are in public service departments, the politician fails to be a satisfactory executive head. Moreover there is hardly a municipal department in the country which is free from the blighting burden of financial mismanagement. It seems to be the fate of municipalities to have their financial affairs conducted in such a manner that a business man who for the first time becomes acquainted with them is horrified. The average public service department conducted by city officials, in case it is revenue-producing, has to pay all its surplus into the public treasury. This is a financial crime of the first order, since the earnings of such a department are diverted from the consumers, who may be but a small fracton of the total population, for the benefit of the whole body of taxpayers. Every competent business man who has ever investigated and reported on municipal financial methods is a strong critic of this almost universal practice, yet it seems impracticable to uproot it from its firm hold. As a result, it is not only impossible to reduce rates for the benefit of the consumers who are paying for the service, but it is even impracticable in many cases to pay out of the earnings of the department the sums necessary for repairs and extensions.

Beyond question this subject of municipal ownership is fast becoming one of the most important in the country. It deserves particularly careful attention for two reasons, the first being that it affords a popular method of appealing to prejudice followed by political demagogues; and, second, there is a feeling among a good many students of governmental methods that the American system of control by popular vote is likely to receive its most severe test in dealing with such a subject. In Germany, where paternalism has always been a feature of government and those who are engaged in municipal service are highly respected, municipal executive offices attract a very fine class of men. There are a good many people, believers in municipal ownership as a theory, who nevertheless hold that the Anglo-Saxon temperament, modified by a mixture of characteristics from the lower classes of all Europe, is unfitted for the municipal management of undertakings so directly affecting all the people as street railways, electric lights, markets, water-works, and the like. Just how far this pessimism is warranted seems to be uncertain at the present time. There are some cities in the country, notably in Massachusetts, where the State regulation of public utilities is very strict, in which the municipal management of public lighting has been a marked success. Moreover, the State control of the extension of railways and other public services by private corporations has prevented unnecessary duplication of facilities, and has practically created for existing companies the monopoly of their business which the committee so strongly and properly considers essential for the successful administration of any public service, whether under municipal or private control. As the matter stands to-day, The Engineering Record believes that a fair statement of conditions is that in some cities it would be extremely dangerous to place all public services under municipal control, and turn over to politicians the direction of their vast body of employees. On the other hand, there are cities where such public service operations might properly be conducted by the city authorities, for the standard of municipal business methods is a high one.

The Water-Works Situation at Atlanta.

The situation of the water-works of Atlanta, Ga., during the past month has been an interesting one, particularly to those who believe that the successful administration of municipal departments depends upon divided control. Without going into the detail of all the troubles, it may be stated in brief that the water supply has at times been deficient in quantity and poor in quality. As a rule it is excellent and the trouble naturally aroused serious protest from the consumers. The affairs of the water department are under the charge of a general manager, a water board and the city council, the measure of responsibility of each being something which apparently few people know anything about. The manager of the department, Colonel Park Woodward, is one of the best known superintendents of such works in the country. The development of the plant and its effective service have been under his special care for years, and that he has served the city well is thoroughly well recognized among water-works men. Probably some members of the city government do not understand how carefully all details of water-works affairs are watched by those interested in them in other cities. Some of these critics, among insurance interests particularly, have a very good reason for keeping closely in touch with water-works management throughout the country and have special facilities for comparing the results obtained in different cities. These people naturally rubbed their eyes when an attempt was recently made to shove on the shoulders of Colonel Woodward the responsibility for the recent troubles with the Atlanta supply. They have known considerably better, apparently, than a good many people in the city that the situation at Atlanta was such that, unless the recommendations repeatedly made by the manager of the works were carried out, the consumers would be subjected eventually to serious inconvenience. Now that the inevitable has happened, although its coming was foretold in the reports of the general manager of the works, the shuffling haste with which those responsible for the mud in what little water was supplied to Atlanta recently, endeavored to fasten the blame for the trouble on the shoulders of others was just a bit ludicrous.

As a city develops and its population increases there is an increased demand for water as well, a fact so self-evident that it is surprising it is overlooked so often by city fathers. It certainly was overlooked or neglected in Atlanta for a good many years. In 1902, Colonel Woodward sent a special report to the water board in which the necessity for various improvements, including an additional water main from the river which is the source of the supply to the reservoir, was very strongly stated. In each subsequent annual report he also called attention to the matter, and the president of the water board drew the attention of the city council during the same time to the subject and urged that money be appropriated to carry out the work. Owing to the lack of this main the city finally was supplied with very poor water, and a few months ago, practically five years after the construction of the main had been strongly urged by the manager of the water department, money for the work was finally appropriated.

The pumping capacity of the water department is short. There are two 10,000,000-gal. pumps and one 15,000,000-gal. pump available for use. During the day it is necessary to run one of the 10,000,000-gal. pumps and the 15,000,000-gal. pump. If the larger one should break down, the two smaller pumps could not give the necessary pressure for fire and domestic service. The situation was brought to the attention of the city council and some of the members of that body recognized its gravity and endeavored to secure a new pumping engine as quickly as possible. Bids for such an engine were opened early last May, when it was discovered that the son-in-law of one of the councilmen had put in a tender for a centrifugal pump driven by a steam turbine. This was a new combination and at the request of the bidder offering it, Mr. D. H. Maury, of Peoria, was retained to investigate the merits of the proposition. This engineer is known to look with favor on centrifugal pumping machinery, nevertheless he rejected the proposition on the ground that the combination was an experiment and it would be hazardous for Atlanta to make it. His report was discussed and he was again retained to investigate the matter, but further examination convinced him that the proposition was not one adapted for the situation in Atlanta. The party making the bid finally suggested that it would be desirable for a committee of the council to visit Brooklyn, where there was such a pumping unit in service, but it was found that the unit was not ready for operation and the trip was called off. Whether a new pumping engine has yet been ordered, this journal cannot state, but the long delay in placing the order is a thing that deserves considerable study by those who urge that the successful management of a water department by public officials demands divided responsibility.

A special committee was appointed to investigate the cause of the muddy water recently and it recommended the construction of settling basins at the river side, where the water could be partially clarified before it was pumped into the main reservoir. This recommendation was satisfactory to the authorities in charge of the water-works and the general manager was asked why it should not be adopted. He very promptly called attention to the fact that two years ago he suggested the necessity of such works. Apparently this suggestion had been entirely overlooked and, like a good many other recommendations of importance made by water-works superintendents in other cities, it was allowed to lapse into the limbo of forgotten things. It is merely one of many details which go to show that there is no inherent virtue in municipal ownership which makes it practicable for several different parties to have control of the same undertaking, and that the sooner such departments are placed in the charge of a technically competent administrator, the better will be the service the public receives. There is no reason why the water board of Atlanta should not be in complete control of the water-works under its charge, receiving enough compensation for its water to enable it to be financially independent of the city council, just as if the service were rendered by a private water company. Under such a plan the works could be conducted in a business-like manner, and there would be a better chance of improvements being made when necessary to avoid troubles like those that have recently occurred.

Notes and Comments.

The Enormous Covered Reservoir which is under construction at Honor Oak, Camberwell, for the Metropolitan Water Board of London, contains a number of unusual features in the design of the walls. The reservoir will contain about 70,000,000 gal., and is divided by two intersecting interior walls into four basins, with a gatehouse for all the basins at the intersection. Here the connections are so arranged that each basin can be operated independently of the others. A part of the reservoir is an excavation, and here the walls are arched in plan, while the outside walls elsewhere are of ordinary section for retaining walls. The central walls are also arched in plan, 6 ft. thick at the center of each bay and 10 ft. at the heavy brick buttresses which separate the bays. These buttresses are likewise used along the outer walls, and are carried up to form part of the roof supports. The floor is made of inverted arches, supporting at the high points brickwork piers 14 ft. high, which carry the jack arches for the drum vaulting of the roof. The brick used in the work were made from the clay excavated from the site.

The Gas Situation in Hamilton, Ohio, concerning which editorial mention was made in the issue of this journal for June 22, has undergone further change. The city council has passed an ordinance granting a franchise to the Columbia Gas & Electric Co. to furnish natural gas to the city for a term of 25 years. The company is to lease the mains of the moribund municipal plant, maintain them, and make necessary extensions, the rental being stated as $5,750 per year. Natural gas is to be supplied by Jan. 1, 1908, the price to be 30 cents per thousand, or 60 cents per thousand for manufactured gas, should the natural gas supply fail. Meanwhile the plant of the Hamilton Gas & Electric Co., which supplies all the gas now used in the town, has suffered considerable damage from a tornado, which on July 6 demolished the condensing house at its by-product coke-oven works, and put this division of the plant temporarily out of operation. The supply of gas, however, has been kept up with but a few hours' intermission, an auxiliary water gas plant being called into service in the emergency. The repairs to the coke oven plant will be completed, it is stated, within ninety days from the time of the accident, and operations will then be resumed as usual.

The Cost of Anthracite Coal is bound to rise steadily and for this reason the expense of operating boiler plants with such fuel in order to avoid the production of smoke is sure to rise with equal steadiness. This assertion has been made a number of times in this journal, and has generally aroused a protest from engineers who hold that the increased cost of hard coal is simply due to a combination of the producers. A few facts based on the experience at a colliery at Pittston, Pa., are worth considering in connection with this subject. The shaft reached a coal seam 10 to 12 ft. thick at a depth of 175 ft., and here coal could be taken out at the rate of 15 tons per miner per day. After this seam was worked out, the shaft was carried down to a second, about 6 ft. thick; then to a third, about 6 ft. thick, and finally to a fourth, only 3 ft. 10 in. thick and 270 ft. below the surface, where one miner can produce but 5 tons per day. Under the present conditions it is necessary to employ three times as many miners to get out the same amount of coal as originally, and also a much larger number of timber men, drivers, runners and masons. It has been necessary to more than double the ventilating equipment and to increase many times the force of men employed in keeping the ventilating system in working condition. The amount of water to be removed has increased from 300 to 3,500 gal. per minute. The wages of the miners have been raised and they are now paid for both coal and rock removed, while formerly they were paid solely for the coal. This list of increased expenses could be made much longer, but enough has been stated to indicate why it is true that the working of the anthracite fields is more expensive now than formerly.

A River Pollution Decision was recently made by the Massachusetts Supreme Judicial Court in Parker v. American Woolen Co., 81 N. E. Rep., 468, which merits attention. The court decides that there is no question about the right of each riparian owner to use the stream in a reasonable manner, according to the usages and wants of the community, which will not interfere with a similar use by the riparian owners above and below him. Many of the uses which properly may be made of the water of a natural stream by an upper riparian owner will have some tendency to defile the water, and it is inevitable for controversy to arise concerning what is permissible contamination. The court rules that no riparian owner has the right to use the waters of a natural stream for such purposes or in such a way as will materially injure it for the use of a lower riparian owner, or to add to it substances which will make it unfit for use. Owing to the self-purification which takes place in streams the law will not prevent the discharge of every small amount of polluting substance into the water, although this may be prevented by statute. The court also makes the important ruling that the plaintiff is entitled to an injunction restraining the pollution of the stream, even though this pollution does not now interfere with any use he is making of its waters. The court accordingly ordered a decree to be entered enjoining the defendant "from emptying or discharging or permitting to be emptied or discharged into the brook upon its premises above the plaintiff's premises any acids, soaps, compounds of soaps, or of iron, chemicals, scouring, dye stuffs, sewage or any objectionable substances whatever, in quantities that noticeably or appreciably affect the purity of the waters when they reach the plaintiff's premises, or render them materially less fit for drinking, domestic or other uses at that point than they are when they enter the defendant's premises."

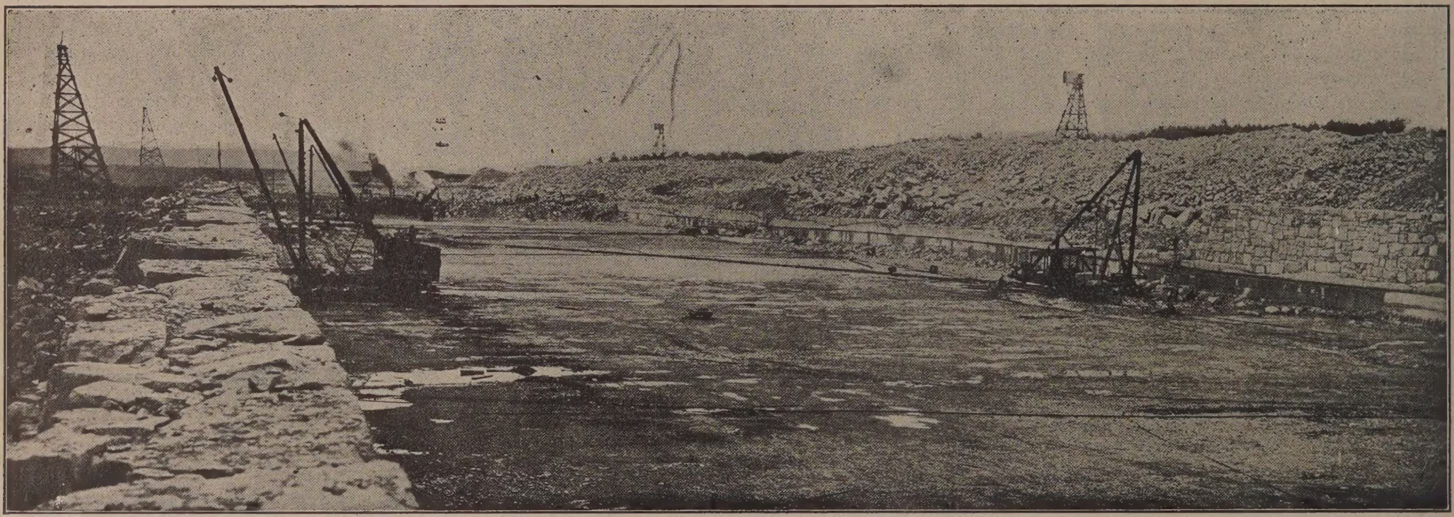
PRIVATE LIBRARY OF FRANK A. RANDALL Case No

THE WEST NEEBISH CHANNEL OF THE ST. MARY'S RIVER.—I.

The improvement of the West Neebish channel of the St. Mary's River, which connects Lake Superior and Lake Huron, was undertaken several years ago by the United States Government and has approached a stage of completion indicating that under normal conditions this extensive work will be finished before navigation closes this fall, or at the latest by the opening of navigation next spring. One of the most serious handicaps to the great volume of traffic passing through this river between Lake Superior and the other Great Lakes will then be relieved. Originally, up-bound vessels plying the St. Mary's River were obliged, after leaving the head of Mud Lake, which is some twenty miles in a direct line from Lake Superior, to pass to the east of Neebish Island and thence follow a long circuitous route around Sugar Island through Lake George and Little Lake George to reach the locks at the Sault Ste. Marie Rapids in the River. The channel along the east side of Neebish Island was improved several years ago to provide a minimum depth of 21 ft. at the mean stage of water in the river. About the same time a channel with a minimum depth of 21 ft. was excavated between the upper end of Neebish Island and the lower end of Sugar Island; this channel was continued on up the river to deep water in Hay Lake, about 10 miles below the locks at Sault Ste. Marie. The new channel thus formed, generally called the Middle Neebish, greatly shortened the sailing distance between Neebish Island and the locks and is used by traffic practically to the exclusion of the old channel, the great fleet of large ore and grain-carrying vessels which pass through the river being included in this traffic.

At the same time the hazards presented to traffic by the Middle Neebish Channel became a serious menace to that traffic long before the latter reached its present tonnage, which during the average open navigation season of 143 days is greater than that of any other inland waterway in the world, having reached a total of over 51,500,000 tons in 1906. Several sharp turns in the Middle Neebish channel, the greater part of which is artificial, are the most dangerous features offered to the safe passage of vessels through it. In the first place the approach to them from one direction cannot be seen from the approach from the opposite direction, owing to the height of the wooded intervening land. On the other hand, the five to six-mile current in the river at this point requires the down-bound vessels to maintain a speed of eight to twelve miles an hour in order to obtain steerageway. Since the larger loaded ore and grain vessels, many of which are over 550 ft. in length and draw in the neighborhood of 19 ft. of water, form the principal portion of the down-bound traffic, it is evident that they cannot be controlled readily at the turns in the channel. Furthermore, the upbound traffic consists mostly of the same large vessels, which generally return to Lake Superior loaded with coal.

The menace offered by these turns and the conditions surrounding them has thus greatly hindered traffic at all times, and has resuled in numerous serious collisions. The extreme care that is observed in navigating this channel, frequently necessitating a vessel to lie at the mouth of the St. Mary's River during the night in order to make the channel by daylight, has, however, reduced the number of accidents to a minimum. To avoid these delays, the schedules of vessels are arranged, when possible, so the channel will be reached during the duy. A large number of vessels, nevertheless, make a night passage through the St. Mary's River, rather than wait for daylight.



Upstream End of Canal Showing Two 1,000-Foot Cableways.

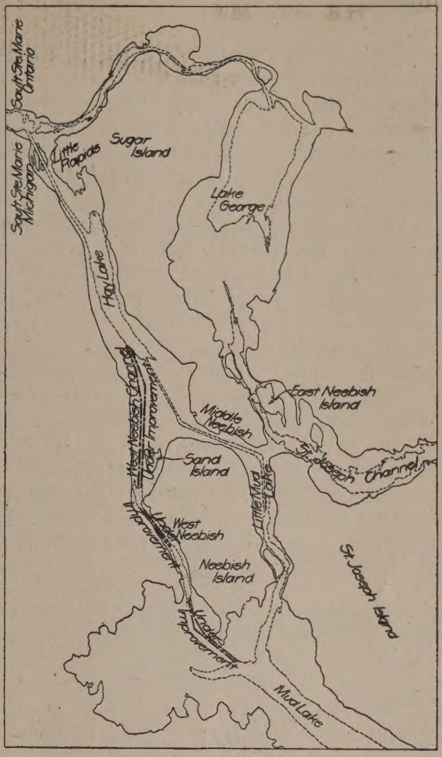
The West Neebish Channel is being excavated to relieve the present situation. It will provide a shorter and straighter course than the Middle Neebish Channel, with fewer turns, all of which can be navigated safely, and will have a minimum depth of 22 ft. below extreme low water. It connects the lower end of Hay Lake with the upper end of Mud Lake, passing between the west side of Neebish Island and the mainland, and has a total length of approximately 13.5 miles. The upper and lower reaches of this channel extending, respectively from Hay Lake to the West Neebish Rapids of the St. Mary's River and from those rapids to Mud Lake were excavated in sand, clay and mud and are finished. The remaining section of the channel, extending through the rapids, with a total length of about 2.5 miles, forms the most difficult part of the whole project, as it has to be excavated almost entirely in solid rock. This section is now rapidly approaching completion, however, the expectation being that, with reasonable success, it will be done before December 1st. After the opening of the new channel on the west side of Neebish Island, the Middle Neebish Channel will be deepened to the same depth of 22 ft. at extreme low water. Downbound traffic will be diverted through the West Neebish Channel after the improvements are finished and upbound traffic will continue to use the Middle Neebish Channel, making a double sailing course in the most difficult part of St. Mary's River, and a practically double course from Lake Superior to Lake Huron.

The new channel has a minimum width of 300 ft., and a clear swept depth of 22 ft. at the lowest stage of the water in the river. The upper reach, extending from Hay Lake to the rock section through the rapids, has a total length of approximately six miles, with only one slight turn. The water along the site of this part of the channel varied from 3 to 5 ft. in depth before excavation was started; the excavation was made through mud and sand with large dipper dredges, the excavated materials being loaded on scows, in which they were hauled to shallow water along the shore line and dumped. The lower reach of the new channel in a distance of some two miles between the downstream end of the rock section and the original river channel had 22 ft. or more water, so no work was required in this length. From the lower end of this natural deep channel to Mud Lake a new channel, approximately three miles long, was dug through the sand, clay and mud which formed the bed of the river. This three-mile section has only one turn, and its channel is widened at this turn to permit easy navigation, as is done at all turns in artificial channels. This section of the channel was also made with dipper dredges from which the material was carried on scows to dumping grounds.

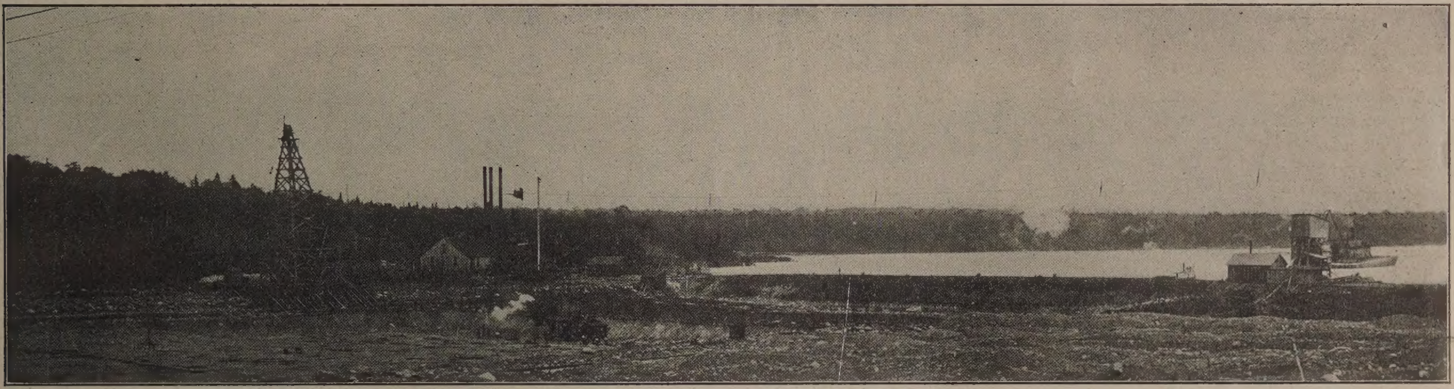
The water in the West Neebish Rapids of the St. Mary's River through which the rock section had to be built, varied from a few inches to several feet in depth and had a current with a velocity of 4 to 7 miles an hour. The bed of the river was a solid ledge of hard Niagara limestone for 8,600 ft. of the 13,300-ft. section included in the contract for this work. This limestone ledge was covered with scattered boulders, varying from small stones to those containing 2 cu. yd. or more. The specifications under which the contract for this work was awarded required that cofferdams should be constructed which would unwater these rapids over a length of about 6,000 ft. of the channel; and that the remainder of the work could be done by dredges. These conditions were changed, however, to permit the construction of cofferdams which would unwater a length of 8,800 ft. of the river through the rapids, the remainder of the work to be done by dredging.

The contract for the construction of the 2.5-mile section of the channel through the West Neebish Rapids was awarded to MacArthur Bros. Co., of Chicago, on April 7, 1904. This company built the large cofferdams required to unwater the portion of the excavation to be done in the dry, and then sub-let that portion of the work, involving approximately 1,700,000 cu. yd. of rock excavation to Messrs. Grant, Smith & Co., of Chicago, of which company Mr. C. H. Locher, associate partner, is manager in charge of the work, and Mr. G. P. Locher is superintendent. The contract for the portion of the excavation to be done outside of the cofferdam with dredges was let to Edward Bros., of Sault Ste. Marie. A preliminary description of the work on this rock section was printed in The Engineering Record for March 2, 1906.

Construction Plant and Methods.—The first work of the construction was to build two temporary cofferdams about midway between the two main cofferdams. These temporary dams were about 1,000 ft. apart at the site of the channel and extended across the river from the mainland to the island, varying in direction to suit the contours of the river bed. They were built in 2 to 7 ft. of water flowing 3 to 6 miles an hour. The construction of these dams stopped the flow of water in the West Neebish Channel of the river, so the main cofferdams could be built in still water, and also laid bare a part of the site of the channel about 1,000 ft. long. In building these temporary dams, which varied from 4 to 10 ft. in height, broken stone and rock were dumped from scows on the line of the dams until the force of the current was broken and the rock-fill carried above the water. Sandy clay was then brought in and dumped on the upstream side of these rock embankments in order to silt up the openings in the latter and produce water tight dams. The dams were completed in July, 1904, after which the water was immediately pumped out of the area enclosed between them and work started on the rock excavation. Two of the four cableways which have been employed in the construction of the rock section of the channel had been installed meanwhile and were placed in service to hoist the excavated material out of the cut and convey it to spoil banks along the channel.



The West Neebish Channel.



Upstream End of Excavation, Showing Part of Upstream Main Cofferdam and Compressor Plant.

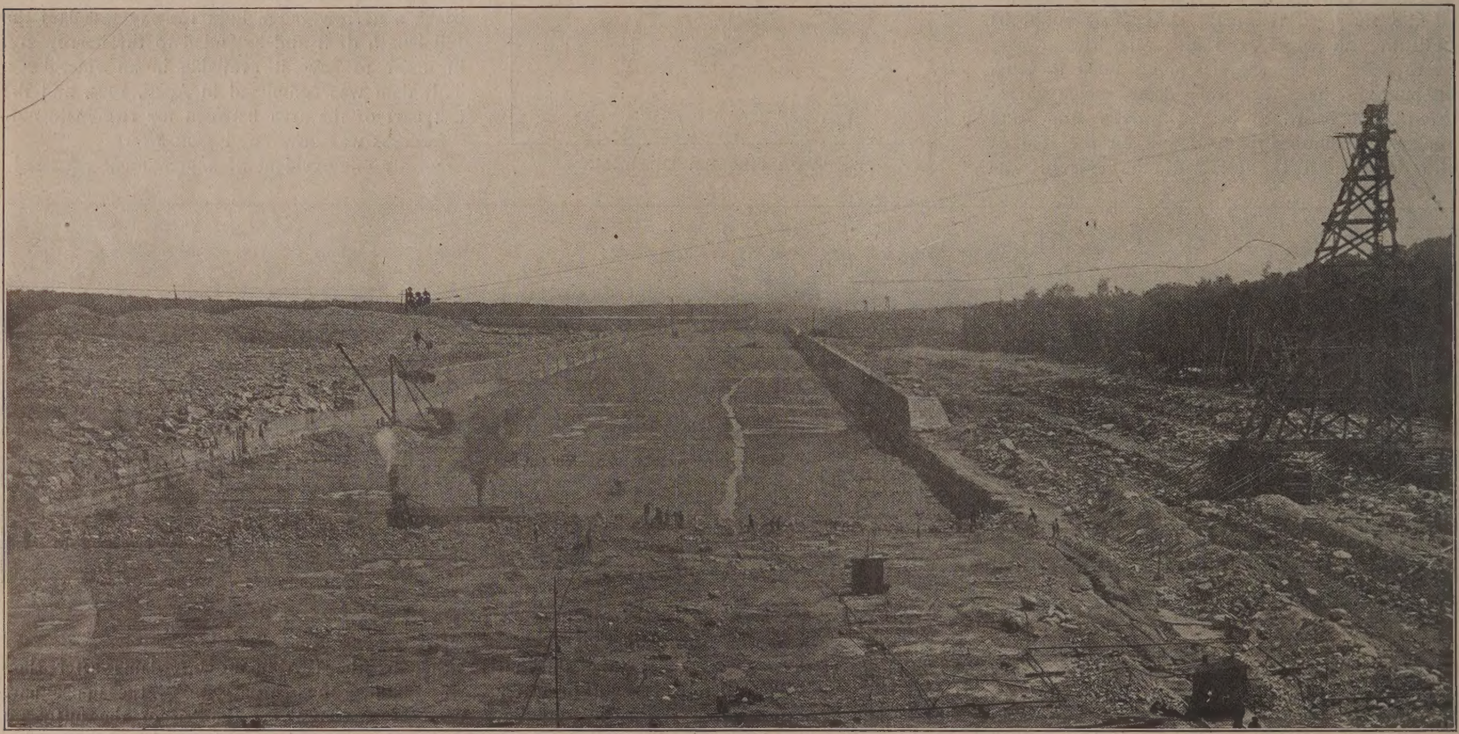
Main Cofferdams.—The two main cofferdams which unwater the 8,600-ft. section of the work that is being excavated in the dry, are structures of unusual magnitude. The upstream dam is L-shaped, and is built in water from 2 to 18 ft. deep; the stem of the ell extends from the mainland across the river at right angles to the channel for 1,200 ft., the leg of the ell continuing down-stream 700 ft. from the outer end of the stem to the shore of the island, making the total length of the dam 1,900 ft. This cofferdam has a minimum width of 8 ft. at its top, which is 7 ft. above the water, and has side slopes on the water side of about 1 on 1½, and of about 1 on 2 on the other side. The other main cofferdam is 8,600 ft. downstream from this one. It has a total length of 2,600 ft., and in plan is arched slightly downstream against the water on that side of it. This cofferdam was built in water from nothing to 26 ft. deep; it has a minimum width of 12 ft. at the top, which is 6 ft. above the water; its side on the water side is built on an average slope of 1 on 2, and the one the other side of 1 on 2½.

The construction of the upstream main cofferdam was started soon after the current of the river had been broken by the temporary dams. Sandy clay and mud excavated by the dredges at work on the adjacent sections of the channel were brought to the site in bottom-dump scows and deposited in place. When the banks thus formed had been carried up until the bottom-dump scows would operate no longer, the materials were loaded on flat deck scows, and handled from these to place in the embankment by a clam-shell bucket on a derrick scow. The final trimming and finishing of the portion of the dam above the water was done with wheelbarrows and by hand. This dam was finished November 1, 1904, after which the water covering the area between it and the upstream temporary dam was pumped out.

The downstream main cofferdam was also built from scows up to within 5 to 6 ft. of the water surface, with materials excavated by the dredges. Soon after this part of the work was finished in the fall of 1904 the river froze over, and the cofferdam was completed through the ice. The rock that had been taken from the portion of the channel on which work was already in progress was hauled on sleds on the ice to a point over the part of the dam already built. This haul was from 4,000 to 4,500 ft., and as much as three cords of stone were carried on a sled at a load. A channel was cut through the ice over the dams in order that the stone could be dumped on top of the bank already built. This dumping was done in such manner as to form a small bank along each side of the top of this base. When these two parallel banks had been carried up to the water surface in this manner, the space between them was filled with sandy clay which contained a considerable quantity of gravel and small boulders, taken from a borrow pit on the mainland. After the space between the two small banks of rock on each side of the top of the lower section of the dam had been filled with the material from the borrow pit the same material was used to top out the dam to its full height of 6 ft. above the water level. The material was hauled from the borrow pit to the dam in boxes on sleds and dumped into place through the ice. After the dam was about finished, a narrow-gauge light railway was laid the full length of it and extended to the borrow pit in order to have it available in an emergency. This dam was completed in April, 1905, and the last part of the area between the two main cofferdams was unwatered soon after.

Only one accident of any moment occurred to either of the two main cofferdams after they were built. In February, 1905, the upper part of a 250-ft. length of the leg of the upstream dam, near the shore where the dam section was light became detached from the remainder of that dam and was carried over into the area which had been unwatered, with the result that that area was flooded. The cause of this accident has never been certainly determined. It is believed, however, to have been brought about by the formation of a line of cleavage in the dam by freezing, followed by a change in the river level which lifted the detached portion and floated it away. This break was soon repaired and since then an open channel maintained in the ice on the water outside the dam has avoided further similar difficulty. This channel was kept open by simply piling brush on it and covering this brush with loose snow; the top and slopes of the dams were also covered with brush to collect snow which prevents freezing and the formation of cleavage planes. Although temperatures of 20° below zero, Fahr., are common in the locality where the work is in progress, this simple expedient forms a line of separation between the main body of the ice and that attached to the slopes of the dam.

Pumping Plants.—The quantity of water which leaks through the cofferdams is very slight when compered with the great length and height of these dams. At the same time, a large amount of pumping capacity was required to remove the water which covered the area enclosed by the dams. The total capacity of the pumps installed is about 40,000,000 gal. per 24 hours. This capacity is required only a small portion of the year, however, but is all necessary in the spring when the snow is melting, or during excessive rains. A berm ditch on each side of the river, extending the full length of the work on the island side and two-thirds the length on the mainland side, diverts the drainage from the land into the river and prevents it reaching the area between the cofferdams. A 12-in. centrifugal pump on the upstream cofferdam handles the principal part of the water in that end of the area enclosed. Two 6-in. reciprocating pumps in this end of the channel cut are capable of removing the water from the latter. Two centrifugal pumps, one a 12-in. and the other a double section 18-in. are installed in a pump house on the downstream main cofferdam to pump the water from that end of the work. Two reciprocating pumps, an 8-in. and a 12-in., are placed in a house on the edge of the channel cut, about midway between the main cofferdams, and pump from the cut into a flume leading to one of the berm ditches. A 12-in. centrifugal pump in a house on the opposite side of the channel near this point is also used while the snow is melting and during heavy rains. The water in the downstream end of the channel cut is removed by three 8-in. reciprocating pumps.



General View of the Construction Work in Progress, Viewed from Upstream End.

General Features of Construction Plant.—The controlling feature of the extensive construction plant which has been installed by Messrs. Grant, Smith & Co., is the adaptability and arrangement of the various parts of this plant to be operated day and night, six days a week, throughout the year. The long and extremely cold winters of the locality in which the work is being carried on required unusually careful precautions in the choice of the equipment to be installed. At the same time, the continued low temperature of the long winters, the large amount of snow-fall which occurs and the great quantity of ice that forms in a work of this character interfered seriously with the progress of the construction. In fact, it is believed that the cost of handling work during the winter months has been 25 per cent. greater than it is during the remainder of the year.

An idea of the climatic conditions in this locality may be obtained from the records of the station of the United States Weather Bureau at Sault Ste Marie. The mean average temperature throughout the year for a long series of years, as given by those records was 39° Fahr. During four months of the year, December, January, February and March, the average monthly temperature for the same series of years was considerably below 32° Fahr., with frequent temperatures as low as 25° below zero, Fahr. Ice forms on open water six months in the average year, reaching a thickness of 30 in., or more during the winter. Added to these conditions some snow falls nearly every day in the winter and the annual snowfall is quite heavy, the ground generally being covered with from two to three feet of snow during the winter. Notwithstanding these extreme climatic conditions the construction work has been handled day and night, practically without interruption, except for Sundays, since its inception. During 1906 the government engineers credited the contractor with 290 working days and nights, without including seven holidays on which work was in progress. This record does not mean, either, that part of a day was counted as a full one, because all of the men are provided with rubber clothing, so work is rarely stopped by rain or snow, the rock bottom of the cut always providing a good working foundation.

The various equipment and plant connected with the work is operated, with a few exceptions, on air furnished from a central compressor plant. Air-driven rock drills are employed in making the large number of blast holes in which the channel cut is excavated. The loosened rock is loaded into large skips by steam shovels, the skips being hoisted out of the cut, conveyed to spoil banks along the side of the channel and dumped in these banks by long-span cableways with traveling head and tail towers. All of this equipment was selected largely on account of its adaptability to work continuously under the trying climatic conditions, which become particularly strenuous at night during the winter.

Compressor Plant.—The equipment in the central compressor plant is as complete as that in most permanent plants of this kind, and was installed with as great care. The plant may, in fact, be considered a permanent one, since it has been operated continuously for over three years. It is in a tight frame building, 40x108 ft. in plan, which is divided by a transverse partition wall into a boiler room and an engine room. The boiler room contains three 200-h.-p. Erie City sectional water-tube boilers placed in brick settings on concrete footings. These boilers are operated at 150 lb. pressure and are each served by an 80-ft. stack. They are supplied with feed-water by either of two Snow boiler-feed pumps, which draw from a Kelly-Berryman feed-water heater. This heater operates on exhaust steam from the auxiliary units in the plant.

The engine room contains two cross-compound two-stage Rand compressors; one of these has 40 and 23-in. air cylinders and a 48-in. stroke, producing a capacity of 4,527 cu. ft. of free air a minute at 90 lb. pressure; the other machine has 17 and 30-in. air cylinders and a 30-in. stroke and produces 1,900 cu. ft. of free air a minute at 90 lb. pressure. The larger compressor is driven by a cross-compound 700-h.-p. Newburg Corliss engine, and the smaller one by a cross-compound 325 h.-p. Hamilton Corliss engine. These engines are operated condensing, each of them being equipped with a Dean jet condenser. Circulating water for the condensers is supplied by any one of three pumps in the basement of the engine room. These pumps also supply water to the feed-water heater from the condenser hot well, or from the river.

All of the various units in the plant are on heavy concrete foundations, which were built with special care. The bed plates of the two main units are placed directly on the concrete, without any cap stones. No trouble from vibration of the engines or compressors has ever been experienced, however, although both compressors are in service 23 hr. a day for at least six days a week.

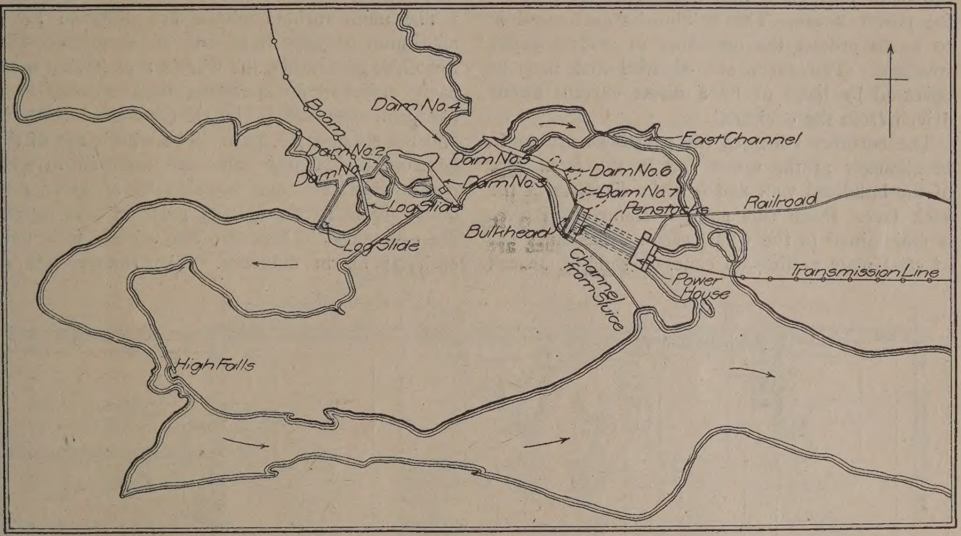
The compressors deliver to an air receiver, 4 ft. in diameter and 18 ft. long, just outside the building, from which air is piped to various parts of the work. An 8-in. pipe line, 6,000 ft. in length, extends along the main-land bank of the river, and is continued an additional 2,000 ft. as a 6-in. line. This 8,000-ft. line is provided with several L-shape vertical joints to provide for expansion and contraction. The line is also laid to drain to drip cocks placed at convenient intervals, in order that the condensation may be removed readily. A second receiver of the same size as the first is placed on the 8-in. line 3,650 ft. from the compressor plant, to provide for sudden demands for air. Taps are made in the main line so branches can be carried to the various plants at any point along the work. A 6-in. main line is also laid to opposite side of the work to deliver air to the engines of two cableways on that side. In all, 35,000 ft. of pipe is required to convey air from the central plant to the different parts of the work. This large amount of pipe is entirely exposed to the weather, but the whole air distributing system has been operated during the coldest weather. Reheaters have to be used on all air-consuming units, however, and in the winter fires are maintained at different points along the supply lines to prevent the water of condensation from freezing and stopping the pipes.

(To be Continued.)

A New Railroad Station designed for handling a large number of people in a short time was completed at East Pittsburg by the Pennsylvania R. R. last spring. It is located at the shops of the Westinghouse Electric and Manufacturing Co., and separated from the latter by Turtle Creek. During the rush hours the railroad carries between 5,000 and 6,000 people, mostly employees of the various Westinghouse companies, who live in Pittsburg and its suburbs. The platforms of the new station have in consequence been made especially wide, and since the passengers as a rule have but a few minutes to wait, the station building has been made small, but the main platforms have been covered throughout their length. The passenger trains are operated on four tracks, the two outside ones being devoted to local trains, which carry the bulk of the traffic. The main platforms are on the outside of these local tracks, and an island platform is located between the express tracks. A bridge over the creek leads to the north local platform, from which stairways descend to a subway leading under the tracks to the other platforms. A masonry retaining wall about 30 ft high forms the bank of Turtle Creek, and in order to get the desired width of platform the station building was built out beyond the face of the wall, overhanging the creek 8 ft. and resting on cantilever I beams.

The Hydro-Electric Plant of the Huronian Co.

The Canadian Copper Co. operates numerous nickel and copper mines, in the Sudbury District of Ontario, Canada, all of which are located on a strip about 28 miles long and not far from the Soo Branch of the Canadian Pacific Ry. The high cost of coal for operating these mines and the smelting plant which is located at Copper Cliff, resulted in the organization of the Huronian Co., in order to acquire the property known as High Falls, on the Spanish River about 28 miles west of Sudbury, and in the spring of 1904 work was begun upon the development of this water power. The plant is located in western Ontario, north of the Georgian Bay.

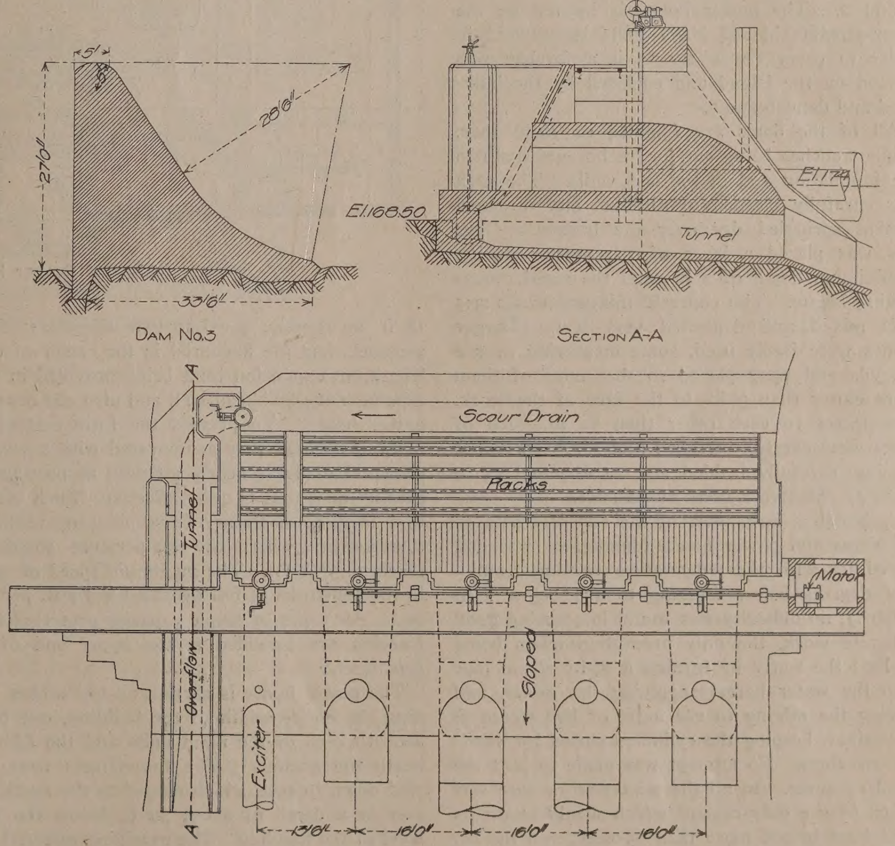


Plan Showing Dams and Power House, Huronian Company.

The Spanish River above High Falls drains an area of 2,150 square miles, over which the annual average rainfall does not exceed 30 in. The minimum gauged discharge from the river is 1,600 cu. ft. per second, which occurred during a season when the average rainfall did not exceed 24 in. Owing, however, to the nature of the covering of the watershed and to the numerous lakes in the upper reaches, the minimum flow is high, being 0.71 cu. ft. per second per square mile.

Above High Falls for a distance of 6 miles the river is a succession of rapids, and it was considered necessary that these should be drowned so as to prevent the formation of frazil, and also to form as much of a reservoir as possible. For these reasons the river level above the falls was raised 18 ft., thus providing a storage basin about 6 miles long and increasing the head from 67 to 85 ft. Immediately above the falls are many rocky islands which break the stream into several channels and finally at the falls themselves an island divides the river into two channels which unite again a short distance below the falls.

On account of the number of small channels between these rocky islands, quite an extensive system of dams was necessary. Three of these were needed to close the west channel, two to close the east channel and two others, joining on to the bulkhead, form the forebay. As the river is much used for lumbering purposes and as large numbers of logs come down each year, provision for their passage was made in dams 1 and 2 by means of a chute to which the logs are guided by a system of booms in the upper river. In order to insure a constant flow over these two dams the other dam in the western channel, No. 3, was made 18 in. higher than the overflow dam in the other or eastern channel. Sluices have been built in two of the dams at an elevation 11 ft. below the bottom of the forebay so that by opening them the water can be lowered if necessary, so as to allow of repairs to the racks and bulkhead. These sluices are also intended to be used to relieve the overflow at high water and are accessible by bridges from the shore end of the dam.



Dam No.3

Section A-A

Dam, Bulkhead and Penstock Inlets.

The first work in the actual development was the construction of the dams, and this was started in September, 1904. The preliminary work had been begun in the Spring of the same year and consisted of the construction of a spur from the Canadian Pacific Ry., and the erection of buildings for housing the workmen and storing material. Dams 1, 2 and 4 were started first and work on them was carried on continuously to completion notwithstanding the severe Winter of 1904 and 1905. About the same time the concrete foundations of the power house were built up above the level of high water so as to avoid delay in the following spring. A log slide and two temporary openings were left in dams 1 and 2, so as to pass the water when it became necessary to stop the flow in the east channel of the river. The latter channel was then closed up by a crib cofferdam against the head of 32 ft. of water. The plan of this cofferdam was in the form of a letter V, each leg abutting on the rocky bank and projecting outward and up-stream. The ends of these legs were finished square leaving a key-shaped space between the channel ends. Accurate measurements were made of this space and a crib of these dimensions was built up-stream, loaded and then lowered with heavy tackle to within a short distance of the proposed location. Its construction was continued until it sank within a few inches of the bottom of the river, and its position was then carefully adjusted so that the current would carry it into the opening between the ends of the two legs. When this had been done the tackle securing it were quickly slacked and the crib slid in place and closed the cofferdam. The face of the entire structure was then double-sheeted and made as tight as the bottom formation would allow and the flow of the river was diverted to the openings in dams 1 and 2. The power house is located on the down-stream side of High Falls Island and in order to carry the water to it a forebay was blasted on the island and enclosed by the bulkhead and dams 6 and 7.

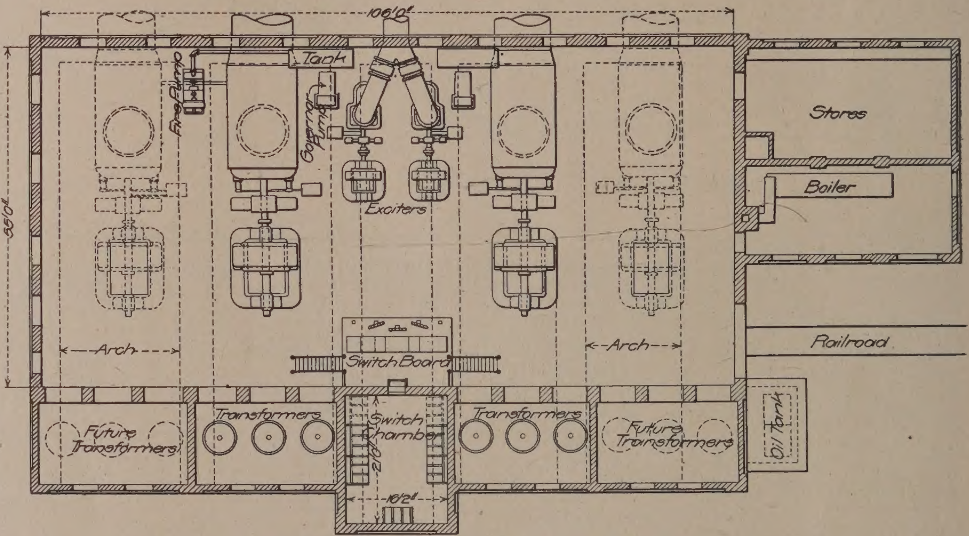
All of the dams are founded on solid rock, ample trenches being cut for anchorages, both in the foundation and in the end walls. The rock was carefully cleaned of all dust and dirt and cement sprinkled dry over the bottom. Over this was placed a layer of 1:3 mortar having a minimum depth of 4 in., and the regular concreting begun. The concrete mixture was 1:3:5 well mixed and deposited very wet. Larger stones were freely used, some measuring over 2 cu. yd., and were placed so that none of them were closer than 9 in. to the face of the work, nor nearer to each other than 12 in. All of them were carefully washed and scrubbed before placing, carefully bedded and so disposed as to form as effective a bond as possible.

Although a large part of the concrete both in the dams and power house foundations was dug in winter with the temperature varying from a few degrees below freezing to 15° below zero (Fahr.), no difficulty was found in securing good concrete work, the only precaution taken being to heat the water by turning a ¾-in. steam pipe into the water barrel supplying the mixer, and during the mixing to use a jet of live steam in the mixer, keeping the cylinders closed by wooden coverings. No attempt was made to heat the sand or stone. In all the winter work care was taken to use only cement which would attain its initial set in not more than 65 min., and the results obtained have been absolutely satisfactory. Even the thin walls used in the sub-station and the blowing engine house were constructed in the winter without impairing the quality of the work.

The dams vary somewhat in height according to the elevations of the rock foundations but are all in the neighborhood of 27 to 30 ft. They vary in length from less than 100 to about 300 ft. in length. The front face in all cases is vertical while the back face in the spilling sections has a ogee curve, and in the remainder of the work a sloped profile having two different ratios.

The steelwork supporting the racks and the entrance cones of the penstocks is built into the bulkhead wall, being carefully bedded and covered with wet mortar. The inlet for the exciter penstocks is separated from the main unit inlet by a concrete pier, and is placed to one side of the four intakes instead of in the center, its usual position. Since the exciter units are placed in the center of the power house with the main units on either side of them the exciter penstock was curved, back of the bulkhead, and carried over the main unit pipes into its relative position in the power house. The bulkhead was housed in so as to protect the operators in severe winter weather. The gates are of steel and may be operated by hand or by a direct current motor driven from the exciters.

The entrance cones of the penstocks are 10 ft. in diameter at the upper end in the front face of the bulkhead wall and 9 ft. in diameter at the back face. From this point a diameter of 9 ft. is maintained to the wheel cases. The tubes are of steel plate resting on concrete saddles, spaced 18 ft. on centers, equal to two diameters of the penstock, and are anchored in the center of their length, an expansion joint being provided in each tube near the bulkhead wall and also one near the power house. To prevent the formation of ice in the penstocks they are covered with a wooden structure made as nearly air-tight as possible, an arrangement that is quite effective. Such protection is quite necessary in a plant operating in this locality where the temperature sometimes reaches 45° Fahr. The maximum speed of water in the penstocks at full overload is 7.2 ft. per second. Air pipes of ample capacity protected from freezing are provided at the upper end of the penstocks.



Plan of Power House, Huronian Company.

The power house is carried on five arches running the entire width of the building, one being beneath each of the main units and the fifth beneath the exciters. The foundations were carried down to rock which dipped to the north and east at a depth of about 30 ft. below the floor level of the building. The overlying material was soft clay and quicksand which gave a good deal of trouble in excavating, though the low temperature at which most of the work was done tended rather to facilitate than retard progress as every opportunity was given the frost to penetrate the earth beyond the limits of the excavation, thus rendering the standing wall secure and enabling the excavation to be carried down about plumb.

The floor arrangement of the power house provides for two exciters in the middle of the room each capable of serving four generators. Both of them are furnished with water from one penstock and each is provided with a hydraulically operated gate valve. The power units, of which there will eventually be four, are arranged two on each side of the exciters. The switchboard is in the center of the building, in front of the exciters, and back of it is the tower in which the high-tension switches are housed. On either side of the latter are the transformer compartments separated from the main room by steel doors. A railway track leads into the power house and an overhead crane commands all the machinery, the transformers being mounted on trucks so that they can be pushed forward under the crane.

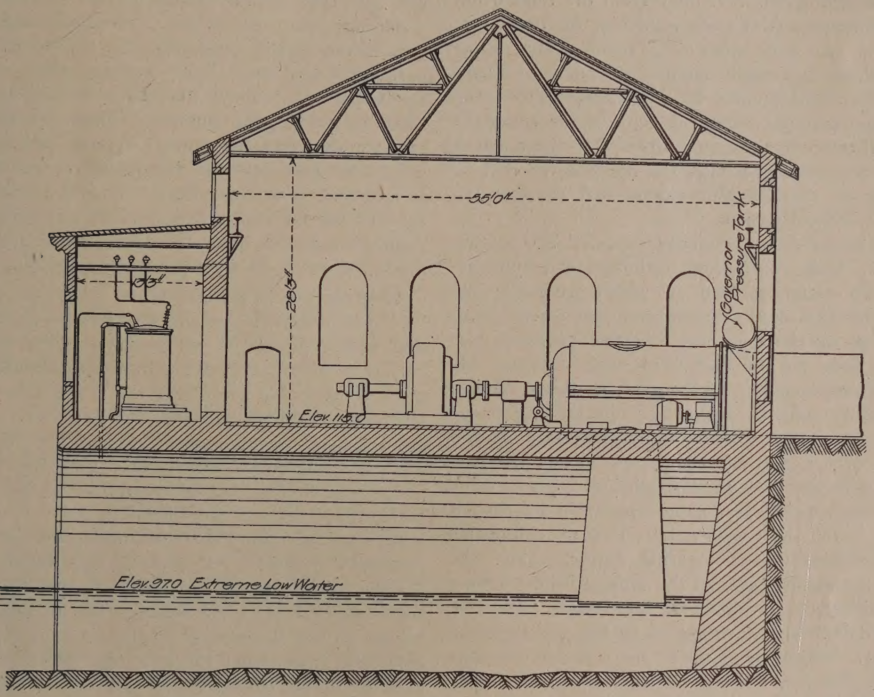
The main turbine wheels are designed for a maximum of 3,550 h.-p., and are connected with 2,000-kw. generators, the wheels thus having a capacity sufficient for operating the generators at 33 per cent. overload. The effective head is 85 ft. and the speed 375 r. p. m. The wheels are of the horizontal shaft type and are enclosed in steel cases, which are split horizontally so as to give quick access to the moving parts in case repairs are necessary. There are two wheels to a unit, each 34 in. in diameter. The runners are of bronze. The thrust bearing which is of the marine type, is located between the wheel case and the generator. The bronze gates are of wicket type being as nearly as possible balanced and are operated by a governor of the Sturgess type. The exciter turbines are similar in design to the main units and are also controlled by Sturgess governors. All of the governors are electrically controlled so that the operator on duty at the controlling desk can start up any or all of the machines. The total full load capacity of the plant is 8,000 kw. The exciters each have a capacity of 200 kw. and are operated at 550 r. p. m.

In testing the generators already installed the loads were obtained by connecting one generator up as a motor, reversed in the direction of rotation, and operating it from the other machine running as a generator. Both machines for the purpose of the test were brought up to full excitation and switched together before starting. The gates on the generator wheel being opened up gradually, both machines came up to speed, the motor, of course, driving its wheel in the opposite direction to normal rotation. When both machines were at full speed the gates on the wheels connected to the motor were gradually opened, thus obtaining practically a water brake load. It was possible by this scheme to get any load required and also to reach any power factor by over or under exciting the synchronous motor, thus determining the regulation of the generators at any power factor. This scheme proves decidedly superior to a water rheostat load which gives only a fixed power factor.

The marble bench board from which the operation of the station is controlled in every particular, including the speeding of the water wheels, was placed on a gallery to give the operator a clear view of the power station and also of the switching apparatus in the tower back of the board. The main switches and all connections between the main units, transformers and transmission lines are distantly controlled so that the only voltages on the bench board are those from the exciter and the operating voltages from the switch control. In other words nothing higher than 125 volts is admitted to the main power station except within the generators themselves which have no exposed parts and the current from which is carried by cables in ducts to the low voltage chamber in the switch tower.

There is a group of three transformers to each main unit, each group being housed separately. The transmission line from High Falls to Copper Cliff is operated at 35,000 volts and is about 29 miles long, running for the most part upon a right of way acquired immediately outside of the Soo Branch of the Canadian Pacific Ry., thus affording ready access from the railway at any point. The line consists of two 3-phase circuits.

The plant was finished in February, 1906, and is now supplying current to five sub-stations located at the mines and the smelters of the Canadian Copper Co., the Copper Cliff station being the farthest from High Falls. The plant was designed by Messrs. Ross & Holgate, consulting engineers of Montreal, Canada, and described by them in a paper before the Canadian Society of Civil Engineers, from which this article has been taken.



Section Through Power House, Huronian Company.

A Curtis Turbine Test was made by the engineers of the Boston Edison Co. on Jan. 29 of this year, the machine being one of 5,000 kw. rating. Under the normal operating conditions the test showed a steam economy of 13.586 lb. of steam per kilowatt-hour. On Feb. 25 of this year one of the four 8,000-kw. turbines at the Fiske St. station of the Chicago Edison Co. was unofficially tested under regular operating conditions, and showed a steam economy of less than 13 lb. per kilowatt-hour. This test, moreover, showed the remarkable result that from 5,000 to 14,000 kw. output the steam consumption varied less than 1 lb. per kilowatt-hour.

The Engineering Organization of the Western Pacific Railway Company.

By George P. Low.

The Western Pacific Ry. is now being constructed from Salt Lake City to San Francisco, its engineering and construction department being under the direction of Virgil G. Bogue, first vice-president and chief engineer of the company. That the construction work may be handled to the best advantage, the line has been divided into six sections in charge of division engineers with headquarters as follows: 1, The San Francisco Division of 93 miles, between San Francisco and Stockton, in charge of Mr. John T. Williams, Oakland, Cal.; 2, the Sacramento Division, 113 miles, between Stockton and Oroville, Cal., subdivided into two sections, the first of 87 miles, between Stockton and Yuba River, under Mr. John T. Williams, and the second of 26 miles, between Yuba River and Oroville, under Mr. Emery Oliver; 3, the North Fork Division, 76 miles, between Oroville and Spanish Creek Crossing, Cal., under Mr. Emery Oliver, Oroville; 4, the Sierra Division, 163 miles, from Spanish Creek Crossing to Deep Hole, Nev., under Mr. J. Q. Jamieson, Clio, Cal.; 5, the Humboldt Division, 227 miles, between Deep Hole and Elko, Nev., under Mr. Charles Harlowe, Winnemucca, Nev.; 6, the Salt Lake Division, 252 miles, between Elko and Salt Lake City, under Mr. T. J. Wyche, Salt Lake City.

The North Fork Division is one of the heaviest pieces of railway building ever undertaken, and it is the purpose of this article to describe the engineering organization under which its construction by The Utah Construction Co., of Ogden, Utah, is being directed. For practically the entire length of the division, the route of the railroad follows up the tortuous canyon of the Feather River, first along the main stream to the junction of its North and Middle forks, which is at a point about 7½ miles by way of the grade northeast of Oroville, thence along the North Fork for 57 miles to its confluence with the East Branch of the North Fork which it follows for approximately 17 miles. It then enters the canyons of Spanish and Spring Garden Creeks respectively and, leaving them by tunneling through the main ridge, reaches the Middle Fork of the Feather River, which it follows through Sierra Valley until reaching Beckwourth Pass.

The North Fork Division is divided into residencies, each about five miles in length, each being in charge of a resident engineer who reports to an assistant engineer. The latter has direction of the engineering work on two or more residencies through various resident engineers, and is responsible to the division engineer. In some cases the work of the engineers is so arranged that assistant engineers are given two instrument men with parties working from the same camp, the men being rated as resident engineers only when they are in charge of a camp. Resident engineers are usually allowed a party consisting of a leveler, chainman, rodman, stakeman, axeman and a cook, but assistant engineers have authority to increase or reduce the number of assistants employed. Where a camp is maintained by the company, expenditures for board elsewhere are discouraged, but when assistant or resident engineers do not wish to board at the company camps, upon application to the division engineer they are allowed $25 per month for board in lieu of subsistence, with proper deduction from this amount for meals which may be taken in company camps. Engineers are not allowed to board their wives and families at company camps, although occasional meals to them are not prohibted.

Assistant engineers are given general supervision of the work to which they are assigned and are responsible for its proper execution. They receive such reports from the resident engineers as are necessary, and these reports, after being consolidated and approved, are forwarded to the division engineer. Assistant engineers are furnished with an advance account with which to defray current expenses which cannot be vouchered. All time rolls, bills of material and estimates made out by resident engineers are sent to the assistant engineers, but to save time weekly progress reports and requisitions for camp supplies are generally sent to the division engineer direct.

Assistant engineers are instructed to give as much personal attention to the details of construction work, where supervision is assigned to them, as is possible, and they are required to assist and direct the resident engineers in every way necessary, as they are held responsible for the proper execution of the work of the resident engineers and their assistants. When necessary, assistant engineers accompany their force reports with letters to the division engineer calling attention to any lack of energy on the part of the contractors or to any disposition to slight or neglect work.

Diaries are furnished to assistant and resident engineers in which all special data pertaining to the work are kept, such as dates of beginning and completing special structures, notes on classified materials, the manner in which contractors handle various kinds of materials and the like. The condition of the weather is also noted, and where camps are convenient to the river, gauges are established and readings taken during floods, these readings being turned in with the final records. The engineers are required to furnish all necessary assistance and information to inspectors detailed to look after special structures and to work in harmony with all other engineers and their assistants. As friction between employees is prejudicial to the company's interests, it is not tolerated.

The following reports are required by the division engineer:

Progress Report.—Every Sunday the resident engineer mails to the division engineer a progress report giving any and all information pertinent to the work which he has in charge.

Contractors' Force Report.—On the 8th, 15th, 22d and last days of each month, assistant engineers are required to make up from data received from resident engineers, a report of the forces of every character employed by the contractors. These forces are counted at least once a day by a time-keeper, who is checked by the assistant and resident engineers as they go over the work from time to time.

Financial Estimates.—Data for financial estimates are compiled and forwarded early enough in the month so that approximately the amount of money necessary to meet pay rolls and vouchers can be provided and forwarded to the division engineers before the last day of the month.

Property Reports.—A complete inventory of all company property in charge of assistant and resident engineers is made in duplicate and forwarded to the division engineer on the last day of each month, and engineers in charge of such property are held responsible for its safe-keeping and care. All camp equipage and instruments received during the month from all sources are entered in the property reports, and reports are also made of the date chiefs of party leave the service or are transferred, and certified to by both outgoing and incoming engineers. If it should happen that the incoming engineer is not present when the outgoing engineer leaves, then the report is signed by the employee temporarily in charge. The new appointee, when assuming charge, also signs it, with any necessary endorsement, and then forwards it to the division engineer.

Time Rolls.—In making out time rolls, the blanks furnished must account for each day worked, as well as for the month's aggregate. In figuring for fractional parts of a month the valuation for a day's labor is figured to the nearest five cents. If time certificates have been issued, the number of the certificate is entered in the proper column opposite the name on the roll. The number of time voucher is also entered by the division engineer or the assistant engineer issuing it. Engineers certifying to time certificates and time vouchers are held responsible for the correctness of the same and their proper notation on the time rolls. Time certificates are not issued unless the employee is leaving the company's service, but when doing so after the rolls have been sent to the division engineer, suitable identification certificates are furnished.

In transferring a man from one party to another, a letter is written by the transferring engineer to the receiving engineer indicating the last day on which time is allowed. In addition to this, a letter of advice is sent to the division engineer's office with the same information, and any failure to do so imposes the penalty of any resultant loss to the company on the party at fault.

Itemized Statements.—The division engineer cautions his subordinates to be systematic in the keeping of accounts, and to call for bills from dealers a few days before the end of the month so they can have these accounts rendered promptly. The practice of having bills sent to the division engineer's office weeks or even days after they should be, is discouraged. Bills for which discount for cash can be obtained are called for when the purchase is made and are sent in at once. All bills must be in duplicate; one being in copying ink or copying pencil. They must show the amount of purchase, the unit price, and calculations checked to the nearest five cents. They are then certified by the resident engineers and forward to the division engineer either preceding or accompanying the subsistence report for the month in which incurred. Receipts are not dated after the last day of the month to which they belong. Board bills must show the name of each man and the dates on or between which the board was furnished. The rate per day or week is also shown, as well as the work upon which the party was engaged during the time. All payments made to contractors are vouchered, and under no circumstances paid directly or entered on pay rolls.

Requisitions.—Where engineering camps are reached by team, the resident engineer, on or about the 20th of the month, makes out a requisition for such supplies as are necessary to last during the following month. The supplies so requisitioned are sent to the storekeeper, after approval by the division engineer. These requisitions do not include farm products, which can be bought near the camp, and standard packages only are sent by the storekeeper, even if a single package will last several months.

Subsistence Report.—This report is made up monthly by the chief of each engineering party subsisted by the company immediately after all bills for supplies furnished during the month have been certified and submitted through the proper channels. An inventory of food supplies on hand after the last meal at the end of the month is taken and entered at actual cost. After deducting this inventory from the total debit, the difference shows the total cost of meals furnished. An accurate account of the total number of meals furnished during the month is kept and the debit divided by this number shows the average cost per meal. A copy of the summary for all camps for the month is sent to the resident engineers that they may see the relative cost of meals at the different camps and correct any waste or extravagance.

Estimates.—The estimates rendered are of two kinds: main contractors' estimates and sub-contractors' estimates, both of which are made out in ink. The main estimates are sent so that they will be received by the division engineer not later than the third day of each month. All the section numbers of the work in charge of any assistant engineer are shown on the sheets in consecutive order, with the limiting stations for each, even if work is being done in but one section; the idea for so doing being to have each quantity which is estimated appear in its proper place in the sheet each month, thus providing the utmost facility in comparing and checking. If only the second page of the estimate sheet shows quantities, the headings on both pages are filled in and both sides are signed by the assistant engineer. Inasmuch as all of the route of the line within the North Fork Division traverses a section of the mountains never before covered by a wagon road, it has been necessary for the company to build one to enable the contractors to carry in materials and supplies. Where work on this wagon road is being carried on adjacent to the line of the railroad, the section numbers corresponding to the main line are shown so as to indicate the portion of the line to which the wagon road is opposite; but, conversely, the station numbers of the wagon road are independent of those of the railroad.

Sub-contractors' estimates are also prepared in duplicate and sent to the division engineer with the main contractors' estimates. They show the work done by The Utah Construction Company as well as by the sub-contractors, and the estimates divide at the section limits in order that they may be checked against the main estimate sheets. The sub-contractors' estimate sheets are complete statements of all work done and are handed to The Utah Construction Co. for its use in settling with its sub-contractors. Station men are not shown thereon.

In estimating quantitites for the monthly estimate, the nearest even 10 cu. yds. is taken when the unit price is $1.00 or less per cubic yard, and fractions of a unit are not used for any material for which the unit price is less than $5.00 per cubic yard. Material for which $5.00 or more is paid per unit is carried out to the nearest hundredths of a unit.

Extra Work.—Any work that it is possible to measure, or which can be covered by the specifications at the contract price, must be shown on the estimate sheets, but when it is necessary to have any extra work done by the contractor's forces, it is carefully looked over and, if sufficient in amount, a time-keeper is kept on the work all the time. A force of half-a-dozen men employed on extra work is considered to be sufficient to justify keeping a time-keeper.

Bills for extra work are made out by the contractor in duplicate and, when approved by the assistant engineer, are sent to the division engineer. These bills are sent in with the estimate at the end of the month in which the work was done, and are not allowed to run along until it is necessary to include more than one month's work on the bill. If the bills, as made out by the contractor, are not in proper form, the engineer in charge must make up the bills, using the contractor's bills as memoranda, the latter being attached to the respective bills and both being sent to the division engineer. All bills rendered by the contractor for extra work which, for some reason, have been disallowed, are sent to the division engineer with a letter giving particulars and the reason for rejection. Bills for extra work show the dates on which the work was done, the number of days work, the rate per day, and the stations between which the work was done, also the distribution and account to which it is to be charged. If in doubt as to the account to which it should be charged, the engineer makes a brief statement on the face of the bill showing of what the work consisted.

Progress Profiles.—These are made on plate "A" profile paper, showing all possible information with reference to the progress of the work. To begin with, the profile shows the ground line, grade line, grade elevation, rate of grade, and alignment. Where revisions of the center line are made, the elevations are shown in dotted lines with new alignment and grades. As the work progresses, the amount of excavation moved and embankment placed are shown in color in their proper places, different colors being used for the respective months. All surfaces are tinted uniformly by mixing enough color at one time to cover all surfaces. The space to be occupied by bridges is left blank until such time as work begins upon them, when the average depth to which piles are driven is shown, and mud sills of framed bents, by short, heavy lines. The stations at each end of the bridge, culverts in place, retaining walls and cribs are shown, care being exercised to color only the portions upon which work has actually been done because of the fact that these profiles are checked with engineers' force reports and estimates. Where the face of a cut is being worked but the center line is not reached, a strip along the ground line proportionate with the amount of work actually done is shown, and where a cut is entirely taken out except for trimming slopes and finishing a small strip is left above the grade line to show that the work is not entirely completed. Fills are not shown completed to grade unless they are of full width.

Standard Progress Drawings.—The division engineer requires that the reports of subordinate engineers shall be uniform in character, and to facilitate the compilation of them in such uniformity, sample sheets in colors are issued to assistant and resident engineers in exemplification of methods to be followed in making calculations and reports upon excavations, fills, grades, tunnels, and, in brief, all features of the work. Excavations, for instance, are classified under three heads, namely: 1, Solid rock, which includes all rock found in ledges or masses of more than one cubic yard and which, in the judgment of the engineer, can be removed only by blasting; 2, loose rock, which includes all bowlders and detached masses of rock measuring more than one cubic foot and less than one cubic yard in bulk, also all slate, hard shale, soft sandstone, disintegrated rock and soapstone that can be removed without blasting, and such hardpan, gravel and bowlder deposits of such consistency as to prevent being ploughed with a ten-inch grading plough behind a team of six horses; 3, common excavation, including all materials not classified as loose or solid rock.

In the sample cross section furnished to the men, solid rock is outlined in brown, loose rock in green and common excavation in yellow, the sectional areas of each being shown with the dimensions from which the calculations are made. Preparatory to making these sections, of course, the work is cross-sectioned, the quantities are calculated and the results then placed on profiles in order to determine where the cuts are to be hauled and where it is safe to borrow, putting doubtful borrow off until the cuts are out. Waste and borrow are not allowed where they will be detrimental to the completed road, and when allowed the borrowed material is not estimated, but any overhaul which might become due had the cuts been hauled, is estimated.

All classified excavation material is measured. and percentages are resorted to only when conditions preclude actual measurements, but solid rock is always actually measured, the results being positively shown. In cross-sectioning the work sections are taken at every station, and wherever any breaks occur in the center or side lines of the excavation, grade points are set on either side of the roadbed where it enters the cut, taking cross sections to each point. The distance out from the center is marked on all grade points and the full station number is given on the back of every cross section stake. On curves, the cross sections are not more than 50 ft. apart.

All work of every character, including ditches, changes of channels, width of clearing, and culvert and trestle excavations, is staked out. As fast as the work is cross-sectioned, the notes are copied into a record book and the original is sent to the chief engineer's office. The cross sections of the work are platted on cross section paper, all figures being put on and classified materials shown in colors as above detailed. The method of average end areas is used in determining quantities.

Construction.—All excavations for retaining walls, cribs, or masonry structures are sunk to a solid foundation, which must receive the approval of the assistant engineer before the structure is started. Where practicable and within the limit of haul, material excavated from foundation pits is placed in embankments in order to reduce borrow quantities accordingly. All loose or dangerous rocks are removed from rock bluffs beyond the regular excavations before commencing to grade the cuts opposite them, this work being paid for by force account.

Embankments are made so as to be full 16 ft. in width after they have become thoroughly settled, and to secure this result, shrinkage is provided for by adding to their height or width as follows: Casting or wheelbarrow work in earth, 12 to 15 per cent.; cars in earth, 10 to 12 per cent.; cars in gravel, or fine rock, 5 to 10 per cent.; wagon work, 7 per cent; scraper work, 5 per cent, and rock fills, 3 per cent. The contractor is informed of the amount of shrinkage which he will be required to put on before he completes roughing-in the embankment. Where the embankments are short or are side fills so that the profile grade must be maintained, the shrinkage is provided for by widening the embankment from the slope stakes up, the shrinkage being made up in the ballast wherever necessary.

The contractor is required to bear the cost of the entire excavation and removal expenses, including loading, unloading, transportation and deposit, provided the haul does not exceed 600 ft.; for excess of this haul, he is paid an additional price per cubic yard per 100 ft. up to the limit of 2,000 ft., or an overhaul of 1,400 ft. All overhaul in excess of the last distance is subject to additional compensation for the additional distance.

Mass Diagrams.—To secure uniformity in estimating overhaul on excavated material, mass diagrams are used in accordance with a sample form. These are made on plate "A" profile paper, laid out to the horizontal scale of 200 to 400 ft. to the inch, the vertical scale being 200 to 2,000 cu. yds. to the inch. In the sample diagram given the men, one cubic yard of solid rock is taken as equal to 1.6 cubic yards. in embankment, but the amount of swell or shrinkage is determined on the ground by observation, the points between which cuts and fills actually balance and the ratio of shrinkage or swell determined being noted. It is found that earth placed in embankments under the blows of horses' hoofs usually shrinks so that 100 cu. yds. of cut will make about 90 cu. yds. of fill, and that rock taken from cuts usually swells 50 or 60 per cent. The quantities used are taken from the cross section notes as they appear in the record book, but when cut and fill both occur within the limits of a station, only the net quantity is used. As shrinkage and expansion affect the distances but not the yardage hauled, the latter is always measured and estimated in excavation.

In making the overhaul diagram, the total yardage, both in cuts and in fills, both ways from grade points is summed up to the various cross section stations within the limits of haul. These stations are platted on a horizontal line to the scale chosen. The total yardage from the grade point to each station is then platted vertically at that station to the yardage scale chosen and connecting lines are drawn. Then a straight edge is moved upward until it measures between the cut and fill lines the proper yardage above 600 ft., at which points horizontal lines are ruled in and the quantities picked off on the vertical scale.

The haul diagrams from which final estimates are made are turned in with the final cross section notes and become a part of the permanent records. The haul is estimated separately for each cut and is paid for on the section from which the excavation is made. A record of the work is kept in ink in a special book provided for the purpose. It shows the station, ground and grade, elevation, cross section, areas, cubic yards, and classification. Surface elevations are shown across bridge openings so that all the necessary information for making a profile may be included. The cross-sections of solid rock or other classified material that can be shown by cross section notes, also all measurements of borrow pits, foundation pits, clearing or riprap, are shown at the end of each section. The measurement for side ditches are given in the remark column, as well as the quantity placed, with the cut to which it belongs. Each section is kept complete by itself and shows at the end a summary of all the quantities for which payments are to be made. The final estimates are made from these summaries. Diagrams for the overhaul from each cut, showing the estimated amount of overhaul for which payment must be made, are attached, as are also diagrams of all special structures, such as cribs, retaining walls, bridge piers, and the like, and bills of material for all culverts with the station at which they were constructed.

Another typical instruction sheet describes in outline the methods to be followed in computing and reporting upon the excavation areas of standard tunnels, as well as of determining the quantities of material necessary for their proper lining. In reality, this sample sheet presents a 63-ft. length of the 806-ft. tunnel known as Tunnel No. 4, which pierces Cape Horn, on the Feather River, about 2½ miles above Oroville. In it the neat section is outlined in red; its enlargement necessary to admit timbering is represented in the space included between red and yellow lines; the overbreakage consequent upon this enlargement is included between yellow and black lines, while the dry packing includes all packing material which is placed by hand outside between the lining or lagging and the line of overbreakage.

Organization Chart.—An organization chart has been prepared, primarily for the information and guidance of clerical forces, which has proved to be of such general utility that its form has been almost universally adopted by the division engineers of the system. In it the engineering organization of the North Fork Division, so far as officials are concerned, is shown at the left, with Mr. Oliver as division engineer; Messrs. H. D. Gradon, A. C. Koogle, Geo. S. Kopp, W. F. Webb, J. A. Hutchinson and A. H. Thomas as assistant engineers, and Messrs. J. F. Miller, C. E. Zeigler, J. R. Howell, W. A. Rowe, C. C. Derby, H. V. Klippel, R. C. Clifford, B. B. Boyd, W. W. Joliffe, R. L. Bryant, and John Tarish as resident engineers. The chart shows the rank of each engineer on the division, with the residencies or residency, or stations to which he is assigned. In the next column appears the numbers of the surveying stations along the route, opposite which are the respective mile posts as surveyed from San Francisco, and between these columns is a double line indicating the wagon road and its bridges, its finished portion being colored red. Following the mile post figures, is a column showing the locations of the various tunnels with their designating numbers and comparative lengths, after which is a line indicating the railroad as sectioned off by contracts, the work opened up by the main contractor and each sub-contractor being given a distinctive color. The scope of each of the four main contracts into which the construction of the North Fork Division has been divided is then shown on the chart, following which the boundary line of the two counties traversed is indicated. Geographical names of points, bars, creeks, etc., occupy the ensuing column, which is followed by a line on which is delineated the rate of grade for the finished road. The locations of water tanks and fuel oil stations are projected from this grade line by means of open circles and black discs respectively. For convenience, the various residencies are again shown, as is the line of the road on which are located the various bridges, tunnels and sidings, with specific information concerning each. The river with its tributaries, the locations of steel bridges and river crossings, together with an indication of the particular side of the river upon which the road runs, are shown with precision. Finally, the width of the right of way at every point is indicated, as is the name of the owner from whom each particular piece of right of way was purchased.

The Sewage Pumping Engines at the Ward St. station of the Metropolitan works at Boston are each rated at 50,000,000 gal. daily capacity against a head of about 40 ft. They are of the Allis-Chalmers vertical triple-expansion type, with standard steam ends. Each engine has three single-acting outside-packed plungers, and six suction and six delivery valve chambers. In each chamber there are 36 specially designed valves of the flap type, with rubber and canvas seats bolted to brass plates. They are hinged and swing on manganese bronze hinge bolts. A modification of the barometric type of condenser, adapted for using sewage for cooling, is used, the passages through it being larger than those employed with clear water in order to avoid any clogging of them.

The Slipping Resistance of Steel and Brass in Concrete.

By H. Burchartz, Gross-Lichterfelde.

Numerous authors have reported on the adhesive strength or the slipping resistance of iron in concrete, for instance Bauschinger, de Tedesco, Mörsch, Féret and von Bach. Dr. O. Meyer has recently made tests relating to this subject and reports on the results of his tests in the "Oesterr. Wochenschrift für den oeffentlichen Baudienst" substantially as follows:

The objects of the tests in question were: (*a*) Dependence of the slipping resistance of the metal rod in concrete on the condition of surface of the rods embedded in the concrete. (*b*) Relation between the slipping resistance and the kind of material of the reinforcement, whether steel or brass. (*c*) Relation between the slipping resistance and the composition of concrete. (*d*) Influence of the form of the rods on the slipping resistance, 1, when the rod was bent to a hook, and 2, when the rod was split to a fork.

All concrete test-pieces were of prismatic shape and square section measuring 20x20x30 cm. or 8x8x12 in. The rods were placed in the specimens in a longitudinal direction, the embedded length amounting to 20 cm. or 8 in.; the ends of the rods were turned off to a 2½-cm. conical shape, so that the tangential adhesion to the concrete was only fully acting on a length of 17½ cm., or 7 in. When calculating the slipping resistance, the cylindrical part of the rod only was considered, for the influence of the cone can hardly be determined numerically in a suitable manner. For this reason the calculated specific slipping resistance may be taken a little too high.

With the rods bent to a hook at the ends the reinforcement was only about 17½ cm., 7 in., in the concrete, and with those shaped like a fork, the end of the fork was found in this length. It is natural that the values for slipping resistance which were found with these latter tests did not at all correspond with the pure adhesive strength, but can merely be used for comparison with the values found with straight bars. Those values are correct standards, however, which indicate the strain per square inch of the sectional surface of the round rods. The latter figures were calculated in order to get the ratio of load to the tensile and breaking strength of the steel.

The materials used in preparing the specimens were: (*a*) Portland cement purchased without knowledge of the origin, but of a quality already approved for concrete buildings; (*b*) Pit-sand with grains passing a sieve with 4-mm. meshes; (*c*) Gravel from the Danube with grains passing a sieve with 10-mm. meshes; (*d*) Round steel bars; (*e*) Yellow metal with a tensile strength of about 30 kg. per square millimeter, and 14 per cent. extension when breaking; (*f*) well water.

The materials were mixed in parts by weight and the mixtures gauged with 10 per cent. of water.

The concrete samples were tamped with the rod in a vertical position.

The specimens in Table 1 were made on November 10, 1900, at about 90° C., the samples of Table 2 on March 29, 1901, at about 6° C. for air and water. The test-pieces were kept dry and for about a month in a shop and then until testing in a laboratory.

The average results of Table 1 are given in Table 3 and the quotient of the average figures of the slipping resistance of rods with rolling skin and with finished surface are also calculated; it fluctuates between 1.5 and 4.8 so that the rough round steel in comparison with the finished one with smoothed surface shows the slipping resistance to be 1½ to 5 times greater.

This result is in good accordance with that obtained by von Bach, who says in "Baumaterialienkunde," 1905: "If the prismatic shape of the rod embedded in concrete is perfect, the slipping resistance depends on the condition of the surface, and if the prismatic shape changes the same is also true. With turned and smoothed rods the slipping resistance is half of that determined for rods with rolling skin." Our differences are therefore still greater. The average of the proportionate values is 2.7 with our tests. The difference in question seems to be generally greater with small steel bars than with large ones; with mixtures of 1:4 and 1:6 the result is, with 12-mm. iron, 2.3 and 3.4 compared with 1.5 and 1.7 with 25-mm. bars.

The strength of concrete seems to influence the slipping resistance, for the richer mixtures of concrete give greater values than the poor ones. It must be borne in mind that the values obtained with 25-mm. steel bars for a 1:6 mixture do not seem to be entirely free from objection, as several test-pieces were already damaged before the test. Even if the results with this mixture are excluded the relation seems to remain true. In the case of the steel bars with the rolling skin left on them, the slipping resistances are, with decreasing richness of the mixture, 697, 514, 426 lbs. per square inch, respectively, for 25-mm. steel bars, while the 12-mm. bars show a decrease from 900 to 740 lbs. per square inch.

TABLE 1.—RESULTS OF ADHESION TESTS WITH STEEL AND BRASS RODS IN CONCRETE.

	Rod			Adhesion per sq. in.			
Mixture.	Material.	Diameter.	Surface.	Age.	Maximum.	Average.	Tension in Rod.
1:1:2	Steel	1 in.	Smoothed	3m. 24d.	62 lb.	......	1,732 lb. per sq. in.
1:1:2	Steel	1 in.	Smoothed	4m. 13d.	229 lb.	146 lb.	6,362 lb. per sq. in.
1:1:2	Steel	1 in.	Skin	3m. 25d.	616 lb.	......	17,182 lb. per sq. in.
1:1:2	Steel	1 in.	Skin	4m. 2d.	798 lb.	707 lb.	22,294 lb. per sq. in.
1:2:2	Steel	7/16 in.	Smoothed	4y. 7m. 2d.	325 lb.	......	18,886 lb. per sq. in.
1:2:2	Steel	7/16 in.	Smoothed	4y. 7m. 2d.	460 lb.	393 lb.	26,838 lb. per sq. in.
1:2:2	Steel	7/16 in.	Skin	4y. 7m. 2d.	879 lb.	......	*50,836 lb. per sq. in.
1:2:2	Steel	7/16 in.	Skin	4y. 7m. 2d.	920 lb.	900 lb.	53,250 lb. per sq. in.
1:2:2	Steel	1 in.	Smoothed	4y. 7m. 2d.	322 lb.	......	8,974 lb. per sq. in.
1:2:2	Steel	1 in.	Smoothed	4y. 7m. 2d.	359 lb.	341 lb.	10,039 lb. per sq. in.
1:2:2	Steel	1 in.	Skin	4y. 7m. 2d.	518 lb.	......	14,484 lb. per sq. in.
1:2:2	Steel	1 in.	Skin	4m. 16d.	508 lb.	513 lb.	14,172 lb. per sq. in.
1:2:4	Steel	7/16 in.	Smoothed	4y. 7m. 2d.	251 lb.	......	14,768 lb. per sq. in.
1:2:4	Steel	7/16 in.	Smoothed	4y. 7m. 2d.	224 lb.	......	13,078 lb. per sq. in.
1:2:4	Steel	7/16 in.	Smoothed	4y. 7m. 2d.	159 lb.	......	9,301 lb. per sq. in.
1:2:4	Steel	7/16 in.	Smoothed	4y. 7m. 2d.	159 lb.	......	9,344 lb. per sq. in.
1:2:4	Steel	7/16 in.	Smoothed	4y. 7m. 3d.	284 lb.	216 lb.	16,614 lb. per sq. in.
1:2:4	Steel	7/16 in.	Skin	4y. 7m. 3d.	777 lb.	......	44,020 lb. per sq. in.
1:2:4	Steel	7/16 in.	Skin	4y. 7m. 3d.	797 lb.	......	59,782 lb. per sq. in.
1:2:4	Steel	7/16 in.	Skin	4y. 7m. 3d.	531 lb.	......	31,098 lb. per sq. in.
1:2:4	Steel	7/16 in.	Skin	4y. 7m. 3d.	848 lb.	......	48,564 lb. per sq. in.
1:2:4	Steel	7/16 in.	Skin	4y. 7m. 3d.	448 lb.	740 lb.	44,730 lb. per sq. in.
1:2:4	Steel	1 in.	Smoothed	4y. 7m. 3d.	73 lb.	......	†2,031 lb. per sq. in.
1:2:4	Steel	1 in.	Smoothed	4y. 7m. 3d.	193 lb.	......	5,382 lb. per sq. in.
1:2:4	Steel	1 in.	Smoothed	4y. 7m. 3d.	119 lb.	......	3,323 lb. per sq. in.
1:2:4	Steel	1 in.	Smoothed	4y. 7m. 3d.	183 lb.	......	5,126 lb. per sq. in.
1:2:4	Steel	1 in.	Smoothed	4y. 7m. 3d.	97 lb.	133 lb.	2,684 lb. per sq. in.
1:2:4	Steel	1 in.	Skin	4y. 7m. 3d.	182 lb.	......	†5,069 lb. per sq. in.
1:2:4	Steel	1 in.	Skin	4y. 7m. 3d.	244 lb.	......	6,802 lb. per sq. in.
1:2:4	Steel	1 in.	Skin	4y. 7m. 3d.	308 lb.	......	.....
1:2:4	Steel	1 in.	Skin	4y. 7m. 3d.	175 lb.	227 lb.	4,927 lb. per sq. in.
1:2:4	Brass	7/16 in.	Smoothed	4y. 7m. 3d.	229 lb.	......	13,320 lb. per sq. in.
1:2:4	Brass	7/16 in.	Smoothed	4y. 7m. 3d.	295 lb.	......	17,040 lb. per sq. in.
1:2:4	Brass	7/16 in.	Smoothed	4y. 7m. 3d.	230 lb.	251 lb.	13,447 lb. per sq. in.
1:2:4	Brass	1 in.	Smoothed	4y. 7m. 3d.	338 lb.	......	9,358 lb. per sq. in.
1:2:4	Brass	1 in.	Smoothed	4y. 7m. 3d.	388 lb.	......	10,906 lb. per sq. in.
1:2:4	Brass	1 in.	Smoothed	4y. 7m. 3d.	500 lb.	409 lb.	13,944 lb. per sq. in.
1:4:4	Steel	1 in.	Smoothed	4y. 7m. 3d.	182 lb.	......	5,084 lb. per sq. in.
1:4:4	Steel	1 in.	Smoothed	4y. 7m. 3d.	173 lb.	178 lb.	4,828 lb. per sq. in.
1:4:4	Steel	1 in.	Skin	4y. 7m. 3d.	471 lb.	......	13,391 lb. per sq. in.
1:4:4	Steel	1 in.	Skin	4y. 7m. 3d.	379 lb.	425 lb.	10,764 lb. per sq. in.

*Concrete cracked and steel stretched. †Crack in concrete, which was plastered with cement. In the text the mixture of 1 part cement, 1 part sand and 1 part gravel is termed a 1:3 mixture, the 1:2:2 proportions a 1:5 mixture, the 1:2:4 proportions a 1:6 mixture and the 1:4:4 proportions a 1:8 mixture.

TABLE 2.—TESTS WITH BENT AND FORKED RODS.

	Rod		Resistance, tons.		
Mixture.	Diameter.	Shape.	Individual.	Aver.	Tension in rod, lb. per sq. in.
1:4	⅝ in.	Hook	6.96	....	55,806*a*
1:6	⅝ in.	Hook	5.22	6.09	41,890*a*
1:3	⅝ in.	Fork	7.05	....	55,948*b*
1:4	⅝ in.	Fork	6.94	....	55,096*b*
1:6	⅝ in.	Fork	6.96	6.98	55,806*b*
1:3	1 3/16 in.	Hook	8.65	....	39,902*a*
1:4	1 3/16 in.	Hook	8.67	....	40.044*a*
1:6	1 3/16 in.	Hook	8.65	8.66	39,476*a*
1:3	1 3/16 in.	Fork	12.0	....	55,948*a*
1:4	1 3/16 in.	Fork	10.1	....	46,576*a*
1:6	1 3/16 in.	Fork	7.78	9.96	35,500*a*
1:3	1 in.	Hook	11.1	....	32,092*a*
1:4	1 in.	Hook	8.01	....	23,146*a*
1:6	1 in.	Hook	5.79	8.30	16,898*a*
1:3	1 in.	Fork	7.34	....	21,158*a*
1:6	1 in.	Fork	15.70	7.68	45,440*a*

a, Sample burst. *b*, Rod broken and loose; sample uninjured.

TABLE 3.—AVERAGE RESULTS FROM TABLE 1.

		Resistance per sq. in.		
Mixture.	Rod.	I—Smoothed.	II—Skin.	Ratio, I÷II.
1:3	1 in.	146	707	4.8
1:4	7/16 in.	393	900	2.3
1:4	1 in.	341	514	1.5
1:6	7/16 in.	216	740	3.4
1:6	1 in.	133	227	1.7
1.8	1 in.	178	425	2.3
		Average Unit Resistance Smooth Rods.		
		Steel.	Brass.	
1:6	7/16 in.	216 lb.	251 lb.	
1:6	1 in.	133 lb.	409 lb.	

The same difference may also be perceived with finished material, though not with the same clearness. These observations are somewhat in contradiction with those made by von Bach, who says: "Alterations in the proportion of the admixture of sand within certain limits do not considerably influence the slipping resistance."

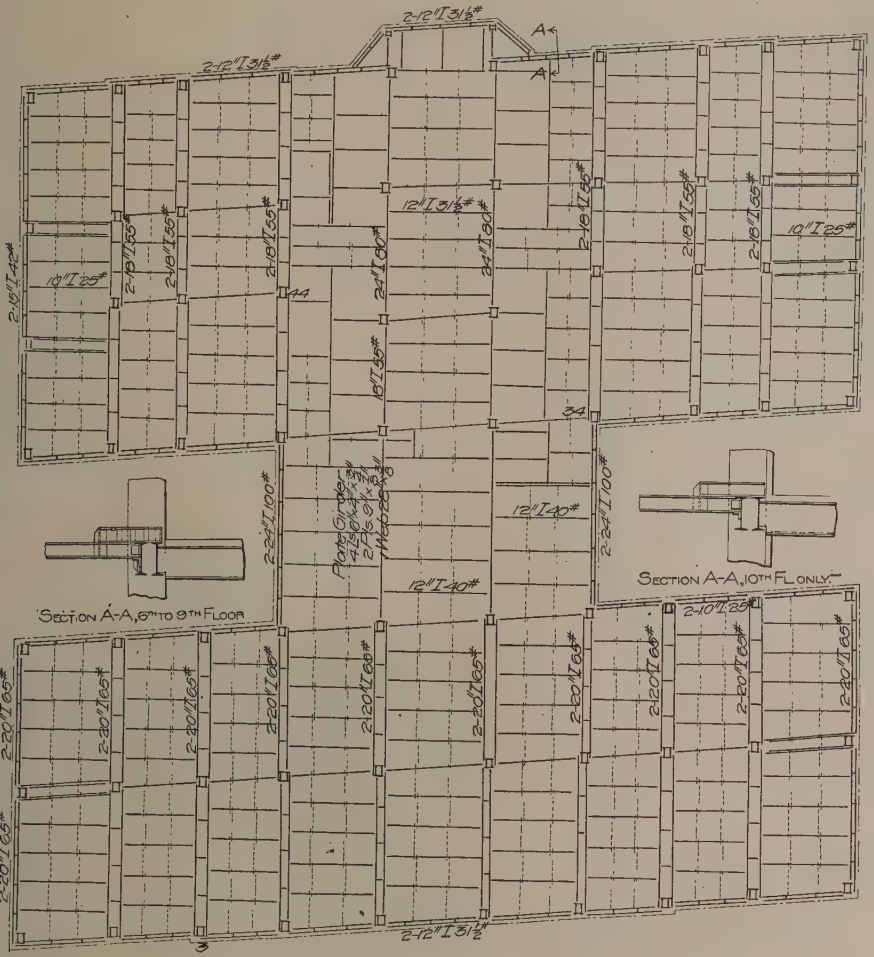
The influence of the thickness of rod has not been distinctly indicated by our tests. Of course the smaller round rods gave more favorable results than the larger ones; but it is the contrary with yellow metal. The observations made with steel contradict those made by von Bach, who found that the slipping resistance decreases with the decrease of thickness of rod. On the other hand, von Bach's remark that with finished rods there are smaller differences, agrees with our results. Yellow metal seems to have greater slipping resistances than steel, which is no doubt due to the softness of the material. This would allow the surface to be roughened more easily and consequently a greater resistance against slipping would result. The finished yellow metal (brass) and steel rods showed equally smooth surfaces at the portion not covered by concrete, so that the finish of the two could be called uniform.

The figures of practical value obtained with the steel bars with rolling skin show on the average a slipping resistance of 651 lb. per square inch; i. e., a value which is somewhat higher than that indicated by Bauschinger. The tests with steel bars bent to a hook and those shaped like a fork showed that the loads required to draw out these rods, are far greater than with straight steel bars with rolling skin. In all cases the concrete body cracked before the rods could be drawn out. With all steel rods the the loads measured, which should properly be called the breaking or bursting loads of the concrete, are so high that, referred to the unit of section of the respective steel bars, they fluctuate between the stretching and breaking load of the steel; in most cases they approach more the latter. The 25-mm. bars form the only exception, which is due to the fact that the surface and section of the rod are as the bases to the squares, the result of which is, with the constant specific resistance, a decrease of the load referred to the section.

The bars split to a fork gave higher figures than those bent to a hook, which result is explained by the fact that the hook-shaped bars when under tension exert only a bursting action on the concrete, whereas the fork also presses the concrete between its two poles thus reducing somewhat the bursting action.

The Hudson Companies' Building, New York.

The terminal station of the Hudson & Manhattan R. R. Co., occupies two city blocks in the congested down-town financial district of New York City, extending from Cortlandt to Fulton Sts., on Church St. It is the terminus of two tunnels under the Hudson River, affording connections with the terminals in New Jersey of the Pennsylvania, the Erie and the Lackawanna R. R., and of all the trolley lines in Jersey City and Hoboken, the passengers from which can enter the subway and elevated lines in New York directly from this station. The station proper has extreme dimensions of about 210x435 ft. and is constructed entirely below the surface of the street in an excavation about 76 ft. deep to the floor of the sub-station.



Typical Beam Plan, 6th to 10th Floors Inclusive.

The massive concrete foundation walls are carried down by the pneumatic caisson process to solid rock at maximum and average depths of about 95 ft. and 80 ft. below the curb and serve to support not only the station but the terminal buildings 275 ft. 9 in. high from the curb to the top of the main roof and 304 ft. high to the tops of the towers. All footings for the interior columns are carried by the same process to rock or hard pan. Below the street, the construction is uninterrupted across the line of Dey St., but above street level it is divided into two large 21-story structures, used as independent office buildings, one on each side of Dey St., which are harmonious in appearance and of the same type of construction and will be operated under the same management and by the same equipment of mechanical installation. The massive sub-structure below street level was separately designed and constructed and may be considered as an independent base on which the fire-proof steel cage superstructures of the terminal buildings will be erected similar to ordinary office buildings.

The two 21-story steel cage fire-proof buildings have brick walls, faced up to the 5th floor with polished granite and Indiana limestone and trimmed above that level to the 16th floor with terra cotta. Above the 16th floor the entire facade is of terra cotta ashlar with brick backing. Together they will contain about 25,000 tons of structural steel, fabricated at the Pencoyd and Trenton plants of the American Bridge Co., about 5,000 tons of terra cotta furnished by the Atlantic Terra Cotta Co., and about 15,000,000 W. A. U. red brick. The floors will generally be reinforced concrete slabs with cinder concrete fill and yellow pine finish. All steel will be enclosed in terra cotta tile, brick or concrete and all wood-work will be fire-proofed. Metal sash and window frames and wire glass will be freely used. The approximate weight of the combined superstructures including the estimated live load is about 165,000 tons.

The dead and live loads per square foot of floor surface are as follows: In the Cortlandt Building dead 85, live 75, total 160 and total roof load 150. In the Fulton Building, dead 95, live 105, total 200 and total roof load 150. Total first and lower mezzanine floor load, both buildings, 250. Stairs and runways, total load both buildings, 270 lb. per square foot. Wind stresses are computed for a uniform pressure of 30 lb. per square foot on the entire exposed vertical surface of the building and the columns loads are calculated with the 5 per cent. allowance taken off from the maximum floor loads at each of the upper floors in accordance with the New York building laws. The maximum unit stress in columns is 13,000 lb. and the minimum stress is about 11,500 lb. The maximum column load is 1,700 tons and the corresponding maximum cross section is 268 sq. in. Above the second floor all columns are made in two-story lengths and the heaviest section of a column in the building from basement to concourse floor weighs about 25 tons. The heaviest girder is in the concourse floor and weighs about 20 tons.

The 213x170-ft. Cortlandt Building, bounded by Cortlandt, Church and Dey Sts., is H-shaped in plan with two symmetrical 32x76-ft. light courts separating wings of 57 ft. and 78 ft. width respectively, which are joined by a connecting structure 65 ft. wide. It contains about 26,000 sq. ft. of rentable area on each of the upper floors. The 156x154-ft. Fulton building is bounded by Fulton, Church and Dey Sts., and is of similar construction with two 48x32-ft. light courts and has about 18,000 sq. ft. of rentable area on each floor. Both buildings are substantially alike and in general symmetrical, but owing to the lack of exact perpendicularity of the intersecting streets on which they are built, none of the angles are right angles and as all of the streets have different grades, great complications in the design and details of the steel work were unavoidable. These, however, related chiefly to the details of connections and are disregarded in the following descriptions.

In the Fulton Building, the first tier of beams in the superstructure is that provided for the pipe gallery at a level of about 13 ft. above the concourse floor and about 8 ft. below the intersection of the Dey and Church St. curb, which has been assumed as datum for all levels referred to in this article. The pipe gallery forms a platform from about 8 to 18 ft. wide extending around two sides of the building under Church and Dey Streets with a floor carried by 6, 12 and 18-in. I-beams supported at one end on the concrete foundation piers and at the other end on beams and channel girders carried by the columns. The main stairway at Dey Street and an inclined runway at Fulton Street, provide entrance and exit respectively from street level to concourse. The concrete slabs of the runway floor are carried on I-beams inclined 1:12½ and supported by plate girders.

In order to provide proper clearance for trains passing through the station, several of the columns have been spaced at wider intervals, below the concourse floor than in the superstructure This arrangement necessitates the employment of heavy triple distributing girders in the Concourse floor to support the 22 upper stories. Some of these girders are supported at one end by distributing and grillage beams on top of the wall column caissons; all of them are encased in concrete and have concrete filling in the interstices between their webs.

The floor designated as the "Lower Mezzanine" is mainly at the street level. It covers the entire area of the building west of the stairway and runway, and is intermediate between the concourse and first floors.

The first floor covers the entire area of the building except where runway and main stairway emerge and a few bays where a greater story height was desirable. East of the runway it is in general at the street level. West of the runway the first floor is intermediate between the lower mezzanine and the second floor, and contains the main elevator hall. In general the above remarks regarding pipe gallery, runway and stairway, lower mezzanine floor and first floor of the Fulton Building, apply also to the Cortlandt Building.

Dey Street roadway between curb lines, including a back fill of about 5 ft., the pavements a
the constant traffic of heavily loaded trucks
wagons, is carried on a system of plate g
and I-beams, which form a skeleton p
about 180 ft. long, and about 27 ft. wi
ported on 14 2-story columns, about 18-

in longitudinal rows parallel to the wall columns in both buildings.

The platform is proportioned for a total load of 1400 lb. per square foot and in the widest panels has 24-in. 90-lb. I-beams about 4 ft. apart, 17 ft. 10 in. long that are web connected to plate girders and carry concrete arches about 18 in. thick.

Two lines of longitudinal girders outside of the ones just described form supports for a portion of the roadway, the sidewalks and several lines of heavy pipes, and are connected to transverse girders. The latter are connected directly to the main columns at one end and to the short columns and main longitudinal girders under Dey Street at the other end. This arrangement keeps the sizes of the girders within reasonable dimensions and provides for the distribution of a part of the weight of the street betwen the street columns and the building columns without involving oblique connections, notwithstanding the fact that the spacing of the columns is irregular and that those in the buildings are not in many cases in the same lines as those in the street.

In the building the floor construction is complete and the 12-in. and 15-in. I-beams are carried directly from the columns and on 24-in. I-beam girders or plate girders. The grade of Dey St., the different levels in the floors, and the irregularity necessitated by the wide stair-ways and their landings make the framing of this tier very complicated.

There are in the lower mezzanine floor of Fulton Building, 58 different girders, of 30 different patterns from 24½ to 40½ in. deep. The web plates are from ¾-in. to ⅞-in. thick and in some cases are made with double 9/16-in. plates. All flanges are made with 6x6 or with 6x4-in. angles reinforced in many cases with 14x¾-in. cover plates. The girders have reactions of from 40,000 to 285,000 lb.

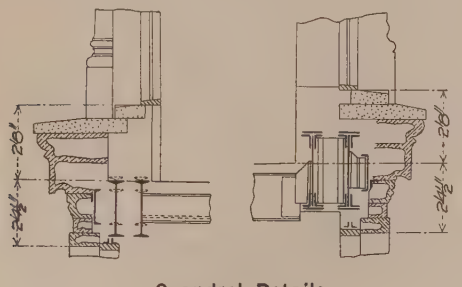
The location of columns and girders in the first floor corresponds essentially with the location in the upper floors, but the floor loads are so heavy that special provision is required for them and the 12 and 15-in. I-beams are carried in most instances by plate girders of which there are 3 types, nearly all of them 30½ in. deep with 6x6-in. flange angles. Owing to the irregularity of the column spacing and the diverging of the column rows there are many offsets in the lines of girders and I-beams and their connections are [illegible] many cases necessarily eccentric and irregular. [illegible] ept on the rear side and in the light courts the [illegible] walls are carried on pairs of 15, 20 and 24-in. [illegible] s. The full size light courts are open to [illegible] et at both ends of the building, but at the [illegible] d 4th floors the beam tiers are extended [illegible] e courts covering the entire rectangle [illegible] the outer walls of the building. The framing of these floors is therefore special in that particular and in a few other minor details, otherwise it corresponds in general to the framing of the 6th, 7th, 8th, 9th and 10th floors, which varies from those above up to the 18th story only in minor details around the bay windows, walls and elevator shafts. In all of these floors the girders are in most cases pairs of 18-in. 55-lb. or 20-in. 65-lb. I-beams symmetrically arranged to give balanced loads on the columns, although where the latter are out of alignment, the variation is provided for in a number of cases by continuing girders of adjacent spans across the opposite faces of the columns and making the other girders of each pair engage the sides of the columns as shown in the diagram.

Above the 13th floor the bay windows disappear and all of the exterior walls carried on pairs of I-beams from 9 to 24 in. deep. The 18th floor framing is special, chiefly in the wall girders which provide for a heavy projecting belt course carried by I-beams or plate girders as indicated in the spandrel sections. In the upper stories of the Fulton building there are 70 main columns from 12 to 36 ft. apart in longitudinal and transverse rows. All of them, up to the 8th floor, have standard rectangular closed cross sections made with web and cover plates and four angles. At the 8th floor the construction changes to rolled channels diminishing from 15 in., to 10 in. in the attic where the minimum size has 12x⅜-in. cover plates. The web plates of the built channels are 16 in. wide in all cases and have cover plates, increasing from 20 in. at the 7th floor to 24 in. in the lower story. All columns have reinforced shelf angles which carry the beams and girders, and are spliced 24 in. above the floor line with horizontal connection angles, a horizontal diaphragm, and a pair of short cover plates, the latter shop riveted to the top of one column and field riveted to the bottom of the succeeding column.

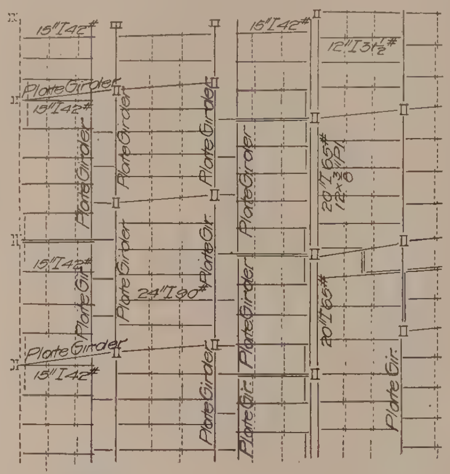
I-beam girders parallel to the column webs are connected by special shelf angles, plates and vertical angles riveted to the cover plates. Those at right angles to the webs are usually riveted to the east and west girders with short connection angles. Girders offset beyond the faces of the columns are carried on projecting cantilever plates properly stiffened. Deep plate girders are additionally connected to the columns and stiffened by auxiliary diaphragms shop riveted to the columns and field riveted to the webs of the girders. The shelf angles which receive the bottom flanges of the beams and girders are reinforced by the milled upper ends of vertical distributing angles which transmit the girder loads to the columns.

In order to avoid wind braces which would obstruct the panels between wall columns and floor beams or would interfere with the requirements of the architectural design, special provision was made to connect both upper and lower flanges of all beams and girders to horizontal angles shop riveted to the columns and thus provide a moment of stiffness equal or somewhat superior to the depth of the girder and insuring a considerable degree of stiffness in the frame work independent of the wall masonry. This construction together with the great mass and inertia derived from the weight and dimensions of the building is considered ample to provide for all wind stresses. The heaviest column in the building is the lower section of No. 44 in the Cortlandt St. Building, which carries 1,700 tons. It is made with four 16x¾-in. web plates, two 24x⅞-in. two 24x½ and two 24x13/16-in. full length cover plates, besides two half-length 24x⅞-in. cover plates and four 6x6x¾-in. flange angles. It is 53 ft. 8 in. long and weighs about 900 lb. per lineal foot. Another very heavy column is in the basement section of No. 34 of the Fulton Building, which carries two deep plate girders in the sub-structure of the building besides the heavy loads from the upper stories. It has four 6x6x¾-in. flange angles, six 16x½ and ⅝-in. web plates and eight 24x¾ and ⅞-in. cover plates.

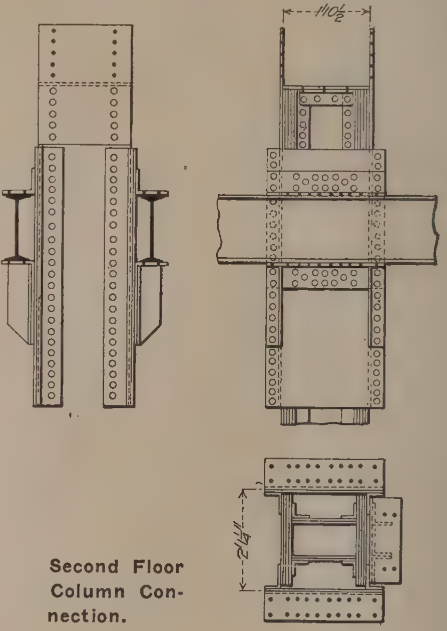
Column 3 between the concourse and the 2nd floor is typical of the upper story columns and the accompanying details show the numerous field rivets through the girder planes and the column connections to provide for the wind stress. The section of column 34 between the concourse and 2nd floors is made with 4x4x¾-in. flange angles, two 16x¾-in. web plates, four 24x11/16-in., and four 24x⅝-in. cover plates, is about 40 ft. long and weighs about 700 lb. per lineal foot. The connections at the upper ends are typical for those of beams and girders throughout the floor and are designed for a maximum stress of 7,500 lb. per square inch on the field driven rivets



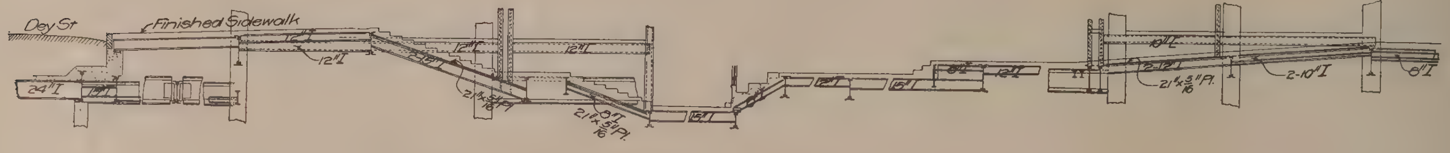
Spandrel Details.



Beams and Girders under Dey St.

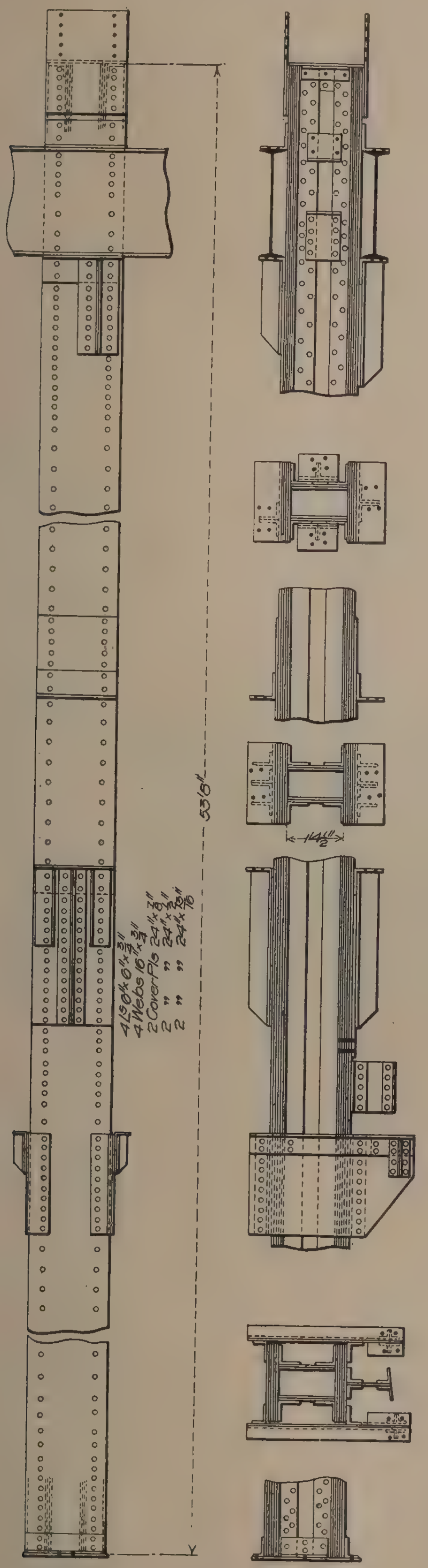


Second Floor Column Connection.



Section of Lower Mezzanine Floor Parallel to Church Street.

Messrs. Clinton & Russell are the architects. Mr. James Dougan, engineer. Messrs. Jacobs & Davies are the engineers for the Hudson Co. The design of the steel work was completed by Purdy & Henderson, consulting engineers, under the supervision of the architect's engineer. The Geo. A. Fuller Co. is the general contractor and the steel work is furnished by the American Bridge Co.

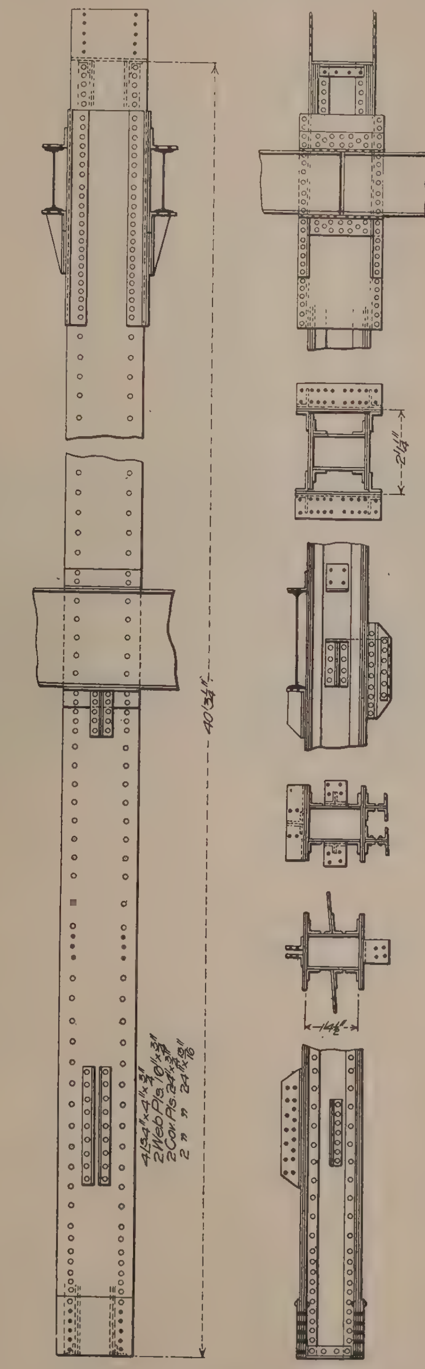


Column 44 A.

Weed Destruction by the use of salt water from Great Salt Lake is being tried by the Oregon Short Line.

A Study of a Ground-Water Supply.

In connection with an investigation of the sources of water supply for Muscatine, Ia., Mr. Wynkoop Kiersted, of Kansas City, who was retained by the city as consulting engineer, prepared a report, which, while it deals with a special problem, is of value to those who have to secure water supplies from gravel strata. Muscatine Island is several miles in width. Its surface is in general above high water and undulates slightly in a direction perpendicular to the course of the Mississippi River. The surface slopes permit little run-off of storm water, hence the rainfall, not evaporated or absorbed by vegetation, reaches the river by flowing through a gravelly substratum. The surface soil is light and porous and readily permits the passage of water.



Column 3 A, Hudson Co. Building.

About 2½ miles below the business portion of Muscatine nine wells were sunk into the gravel down to bed rock or some other impervious material. These were on a line perpendicular to the course of the river. The well nearest the river, termed No. 1, was about 300 ft. from the bank and the others were at distances of 100, 200, 350, 500, 700, 1,000, 1,500 and 3,000 ft. from No. 1, and were numbered consecutively from 2 to 9. No. 1 was 8 in. in diameter, 2, 8 and 9 were 5 in. and the others 2 in. The depth of the wells below the ground surface varied from 46.0 to 57.25 ft. and averaged about 49.5 ft. and the variation in the elevation of the top of the wells was from 115.65 to 120.69, with an averge of 117.82, datum being approximately 100 ft. below low water of the river. The strata through which the wells passed were practically the same in every case and about as follows: 3 ft. of sandy top soil; 5 ft. of clay; 10 ft. of red sand and fine gravel; 10 ft. of coarse gravel; 15 ft. of coarse sand and gravel; 6 ft. of gravelly sand; 2 ft. of sand and coarse gravel; 5 ft. of blue clay; 3 ft. of shaley clay; 6 ft. of sand rock.

A series of observations, taken at intervals of one to five days and extending over a period of nearly two months, was made of the static level of the ground water in the wells and at the same time the stages of the river and the rainfall were noted. During the month of August and the first half of September the river fluctuations were small and there was a correspondingly stable condition of the ground water surface as shown in the wells. The average slope of the surface of the ground water toward the river during this period was 0.8 ft. in 1,000 ft., clearly indicating an under flow toward the river. From Sept. 13 to 21 the river rose a little over 2 ft. and a rise of 2.1 ft. in well No. 1 resulted. A corresponding rise occurred in the other wells, gradually decreasing as the distance from the river increased, until at well No. 9 the rise was only 0.3 ft. The slope of the ground water surface toward the river, though considerably reduced, was still evident, showing that the river water was not flowing into the gravel beds but simply acting as a dam and causing the ground water in the beds to rise and fill the wedge of gravel included between the water table of Sept. 13 and that of Sept. 21. In order for this to happen it was necessary for the water in the gravel to advance toward the river as well as to rise. This advance was estimated by Mr. Kiersted to be at the rate of at least 2½ ft. per day, and it necessarily occurred throughout the entire depth of the gravel bed, about 43.5 ft.

From Sept. 21 to 27 the river rose over 4 ft. and the slope of the ground water surface was completely reversed, so that on Sept. 30 it sloped inland at the rate of 1.13 ft. per 1,000. It was computed that from Sept. 27 to 30 the river water advanced into the gravel beds at the rate of 4.63 ft. per day. From Sept. 27 to Oct. 9 the water table maintained an average slope inland of 0.84 ft. per 1,000 and the computed average rate of advance of the river water for the same period is 2.3 ft. per day. In the accompanying table the results of the computations, involving slope and velocity of flow, are given, v, i and k being the symbols used in the Dupuit formula, $v = k\,i$, where v = velocity in feet per day, i = fall of ground water surface in a given distance divided by that distance, and k = a factor depending on the porosity of material through which the underflow passes.

Results of Underflow Computations.

Interval	v	i	k
Sept. 13 to 21........	2.5	0.0005	5000
Sept. 20 to 21........	1.55	0.000273	5680
Sept. 27 to 30........	4.63	0.00113	4100

The average value of k is nearly 5,000, or the average rate of travel of the underflow of Muscatine Island per day is five times the fall of the ground water per 1,000 ft., indicating a very porous sub-soil.

To secure further information concerning the porosity and continuity of the gravel deposits a pumping test was run at well No. 1, during which readings of the ground water level were taken at the nine wells. The water discharged from the pump was conveyed directly to the river in a wooden trough. The quantity of water pumped was only roughly estimated since it would have been of little use in determining the value of the beds as a source of water supply. A 6-in. centrifugal pump, operated at the rate of about 1½ million gallons per 24 hours, was used. The test was begun in the morning of Sept. 21 and continued for 125½ hours. Between the mornings of Sept. 21 and 26 the river rose over 3 ft. and notwithstanding the steady operation of the pump the level in the wells showed a simultaneous rise. The zone affected by the pumping seemed to extend about 700 ft. from No. 1. The conclusion drawn was that the gravel beds under that part of the island where the wells were driven are practically continuous and so porous as to be favorable for the operation of tubular wells.

The method of development of the gravel beds as a source of water supply depends on the rainfall, the catchment area, the rate of water consumption, the available velocity of ground water flow up to a limit safe for efficient filtration, the depth of the water-bearing gravels, and, it might be said, upon the permissible rate of infiltration from the river were it not for the possible choking of the beds with particles of silt. Although the river will always contribute somewhat to the flow of ground water, the rainfall upon the catchment area is usually considered the main reliance of a ground water supply. At the same time, there should be a large capacity for permanent storage in the gravel bed. The water stored above the low-water mark of the river is an uncertain quantity. If the average stage of the river is low the amount of water in the gravel beds above the low-water line will be small and vice versa. Below the low-water level the beds are sure to be always saturated and will yield water as long as the river and rainfall replenish it.

The amount of rainfall which is available in the run-off by underflow is a larger percentage of the total rainfall in a year of excessive or average precipitation than it is in a year of low precipitation, ranging approximately 50, 45 and 35 per cent., respectively, according to Mr. Kiersted. Taking 45 per cent. of the rainfall for an average year gives 17 in. per year or 813,500 gal. per day per square mile, as the available rainfall at Muscatine. The catchment area supplying the underflow will be on both sides of a line of development and may extend a mile or more on either side of it. For a development having a total width of 2 miles the daily yield per lineal foot is 308 gal. and the velocity 235 ft. per day. When the total width is 3 miles the quantities are 462 gal. and 3.52 ft., and when the development is 4 miles wide 616 and 4.7 are the corresponding daily yield and velocity. The length of the development is not necessarily the same as the length of the constructed work because, as shown by the pumping test, the affected zone may extend upwards of 700 ft. beyond the actual construction.

The computed velocities of ground water flow are based on an available depth of saturated gravel of 35 ft. and an available void space of 25 per cent. of the bulk of the gravel.

Mr. Kiersted recommended that, at Muscatine, the development should be laid out approximately parallel to the river and far enough back from it to insure thorough filtration, probably about 650 to 800 ft. from the bank. The development should be extended enough to insure a slow average rate of percolation. Either the gallery or tubular well system is reported to be feasible at Muscatine, but wells are recommended.

The Reconstruction of the East St. Louis & Suburban Ry. Power Station.

The power station of the East St. Louis & Suburban Railway in East St. Louis, Ill., was built in 1902, with an electrical generating equipment, capable of producing 3,100 kw. Since then this equipment has been augmented from time to time to supply increasing demands for power, and early in 1906 an 80 ft. addition, containing two 1,000 kw. engine-driven generators and 2,000 h. p. in boilers was erected at one end of the original building, the rearrangement of the interior of the latter having been commenced prior to the installation of other new generators.

The original station was built to supply power for the street railway lines in East St. Louis and interburan electric lines in the vicinity of that city. The station building is of brick and is divided by a brick wall into a boiler room, 52 ft. 5 in. wide and 280 ft. long, and a generator room, 56 ft. 9 in. in width and 262 ft. in length. The generator room originally contained two 750 kw. three-phase 25-cycle 13,200 volt alternating current General Electric generators, each direct-connected to a 1,150 h.p. non-condensing St. Louis Iron & Machine Co. Corliss engine; a 1,600 kw. 575-volt direct-current General Electric generator direct-connected to a 2,500 h. p. twin non-condensing Fulton Iron Works' engine, and the necessary auxiliary machines and switch boards. Soon after the station was completed demands were made for power at different voltages from that of the current generated by the units installed, and at present current is being supplied from the station for the following purposes: 575-volt direct-current for street railway service; 500-volt direct-current, over nongrounded double-wire circuits, for power; 2,300-volt 3-phase 60-cycle alternating current for commercial lighting and power; 13,200 volt 3-phase 25-cycle alternating current, which is transmitted to six railway substations and in these transformed to direct current at 550 volts. The power supplied by the original generators was utilized as it came from the latter, transformed to alternating current, or employed to drive motor-generator sets producing current at the desired voltages.

Fans, Stacks and Fan Engine House.

Steam for driving the units in the generator room was furnished by twenty 250 h. p. Heine water-tube boilers. These boilers were set in batteries of four each and were equipped with Green automatic traveling-chain-grate stokers, 12 ft. wide, one grate serving each pair of boilers. Coal was fed to the stokers from elevated bunkers supplied by a gravity overlapping-bucket conveyor that delivered from a hopper under a switch track along one end of the building. Eight additional 250 h. p. Heine water-tube boilers in two batteries have been installed in the addition to the boiler room, and the settings of the existing boilers are being reconstructed. The coal handling equipment will be retained and the new boilers will have grates of the same general design as the existing ones.

When the non-condensing engines driving the three original generating units were purchased they were to be installed in a power house at coal mines about eight miles east of the city, but when the city railway and lighting plants in East St. Louis were acquired by the interests which were erecting the power house, this proposed site of the station was abandoned and the latter was built at the edge of the city. At that time coal could be delivered to the station over the company's tracks from the mines at such a low figure that the non-condensing engines were installed at the new site. Since then the price of coal has increased considerably in the territory in which the station is located, so that in adding to and re-arranging the station, provisions have been made for utilizing the exhaust steam from these non-condensing engines by passing it through low-pressure turbines.

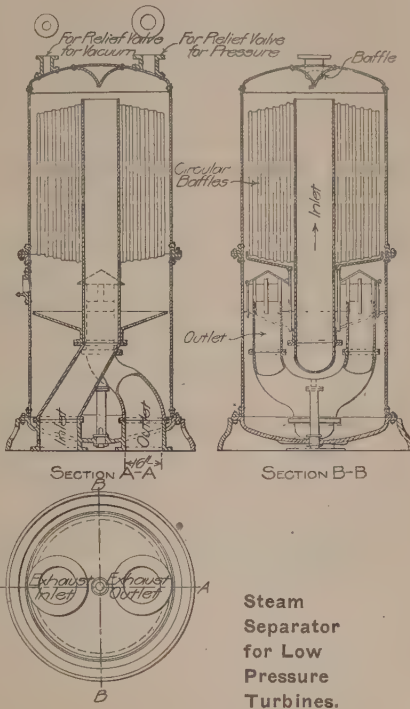
Two new 1,000 kw. engine-driven units have also just been installed, and are designed to operate condensing. In addition to these engine-driven units, two turbo-generator units, one of 800 kw. and the other of 1,000 kw. capacity which operate on exhaust steam, and a 500 kw. link steam turbo-generator have been added to the equipment.

The site on which the station is built afforded such poor foundations that brick smoke stacks could not be erected at a reasonable cost, so five steel stacks, each 6 ft. in diameter and 180 ft. high, were built. These stacks were once partially wrecked by one of the windstorms to which the territory is subject. When the addition to the station was determined on, it was decided to equip the boiler room with an induced draft installation which would provide for the entire plant and to abandon and remove the stacks. This induced-draft apparatus taking care as it does of 7,000 h. p. in boilers, is of unusually large size, and is being built at the same time the other reconstruction work is in progress.

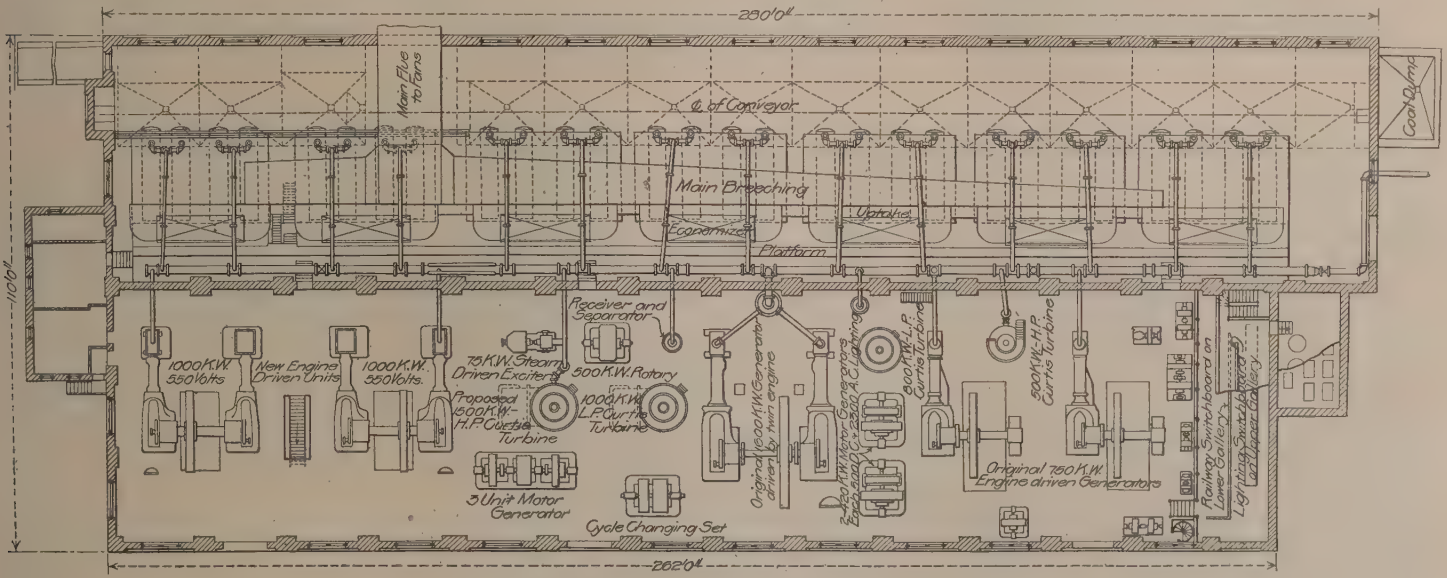
The exhaust steam from the engines driving the 1,600 kw. and the two 750 kw. units is utilized in the low pressure turbines operating on steam at 1-lb. gauge pressure; these turbines are operated condensing, and as about 27 in. of vacuum are obtained, normally, the turbines operate at approximately 14 lb. pressure. An exhaust main 30 in. in diameter is placed in the basement of the engine room and extends the length of the building on the side next to the boiler room. The exhausts of the various reciprocating units are connected to this main and the low-pressure turbines are supplied from it. An 800 kw. 25-cycle 13,200-volt three-phase alternating-current General Electric generator, direct-connected to a vertical low-pressure self-governing Curtis turbine, is placed between the engine of the 1,600-kw. unit and the engine of one of the 750-kw. units, as shown on the accompanying plan. This turbine is supplied with steam from the exhaust main and can be operated independently on the exhaust from the engines of the two 750-kw. units. A 1,000-kw. 60-cycle 2,300-volt alternating-current General Electric three-phase generator, direct-connected to a vertical low-pressure self-governing Curtis turbine, is placed on the opposite side of the 1,600-kw. unit. The turbine of this unit is also connected with the exhaust main, and it is arranged so it can be supplied with exhaust steam from any of the five reciprocating engines. Both of these turbines also have live-steam connections through reducing valves from the main steam header of the station.

The engines driving the two 1,000-kw. units which have recently been installed are equipped with a single Wheeler surface condenser having 3,000 sq. ft. of area; space is also provided for a second condenser of the same size for these two units. One or both of the latter can be operated non-condensing, however, when the exhaust from the three non-condensing units is not sufficient to supply power for the low-pressure turbines.

A specially-designed separator is placed in the connection between the exhaust main and each of the low-pressure turbines, in order to remove the oil and water from the exhaust steam as fully as possible, and to act as a reservoir so that a steady steam pressure may be maintained. The details of these separators are shown in an accompanying illustration. Each separator is inclosed in a cylindrical steel shell, 5 ft. in diameter and about 12 ft. high, which stands on a base on the floor of the generator room. A 16-in. inlet connection from the exhaust main enters this shell at the bottom and extends nearly to the top of the latter, with which it is concentric. The steam is discharged from this inlet against a curved baffle plate in the top of the shell, which baffle directs it downward through a series of cylindrical baffle plates placed concentric with the inlet pipe. These plates retain the impurities and deposit them on an annular diaphragm which is drained to the outside through a blow-off pipe. The steam passes through a narrow space between the outer edge of this diaphragm and the shell of the separator, and then through twin 12-in. outlets into the connection leading to the turbine. The tops of these outlets are capped to prevent any impurities entering with the steam.



Steam Separator for Low Pressure Turbines.


280'0"
110'0"
262'0"
Main Flue to Fans
℄ of Conveyor
Coal Dump
Main Breeching
Uptake
Economizer
Platform
Receiver and Separator
1000 K.W. 550 Volts
New Engine Driven Units
1000 K.W. 550 Volts.
75 K.W. Steam Driven Exciter
500 K.W. Rotary
Proposed 1500 K.W. H.P. Curtis Turbine
1000 K.W. L.P. Curtis Turbine
3 Unit Motor Generator
Cycle Changing Set
Original 1600 K.W. Generator driven by twin engine
2-420 K.W. Motor Generators Each 500 D.C. & 2300 A.C. Lighting
800 K.W. L.P. Curtis Turbine
500 K.W. H.P. Curtis Turbine
Original 750 K.W. Engine driven Generators
Railway Switchboard on Lower Gallery
Lighting Switchboard on Upper Gallery

Plan of East St. Louis & Suburban Railway Co. Power Station.

Connections are made to the separators which will maintain the pressure in the latter, between certain limits, automatically. A relief valve which will prevent the pressure in the separator rising above a pre-determined amount is placed on a connection in the top of the separator. A second relief valve which will prevent a vacuum occurring in the separator is also placed on a connection in the top of the separator. Live steam may be supplied to the separator through an inlet in the side of the latter.

A 500-kw. 60-cycle 2,300-volt alternating-current General Electric generator direct-connected to a vertical Curtis turbine has also been erected, and space for a 1,500-kw. 25-cycle 13,200-volt alternating-current General Electric generator, direct-connected to a vertical Curtis turbine has been reserved in the generator room of the station. These units both operate on steam at about 150 lb. pressure. The 500-kw. unit furnishes power for a commercial lighting and power circuit; the 1,500-kw. unit will supply power for transmission to the street railway sub-stations.

The high and low-pressure turbines are each equipped with a surface condenser. These and the surface condenser for the engines of the two 1,000-kw. units are also supplied with cooling water from the same source, namely, six large cooling towers.

Two of these cooling towers are of the Alberger sprinkler type and the other four are of the Wheeler trough type. The six towers are in a row along one side of the building, each one being on a separate footing. They are built of steel plates and are 19 ft. in diameter to a height of 33 ft. and then 10 ft. in diameter to the top of the tower, 60 ft. above the footing. The cooling water is drawn from the base of each tower, circulated through the condensers and forced back to the top of the towers by centrifugal pumps in the basement of the generator room of the station. A pump is provided for each condenser, each pump being direct-connected to a motor. The two sprinkler-type towers are each equipped with two 8-ft. fans while the four trough-type towers each have two 10-ft. fans. Each pair of fans is connected through a shaft to a motor in the generator room of the station. Each tower is connected to a supply pipe from the city main, these connections being fitted with valves by which the make-up water is supplied automatically.

The condensed steam is pumped from the condensers to a 6x17.5x5-ft. closed hot-water tank in the basement of the boiler room. This tank is arranged with vertical baffles, placed 1 ft. apart, which are open alternately at the top and bottom of the tank. The water is admitted at one end of this tank and passes in a circuitous up-and-down route around the baffles to reach an outlet at the opposite end. The oil and other floating impurities are thus caused to rise to the surface between the alternate pairs of baffles, which alternate baffles are attached to the top of the tank. A blow-off pipe is placed in each of these spaces for removing the impurities from the surface of the water.

The induced-draft system which will replace the stacks that now serve the boilers has a main breeching extending the length of the row of boilers over the tops of the latter and connecting in a space between the old and new boilers to a main flue leading to a fan house that has been built on a separate footing at the side of the main building. Two fans, having impellers 20 ft. in diameter and 10 ft. wide are placed along the side of this house, in steel-plate volutes 32 ft. in diameter.

Each of the batteries of two boilers has been equipped with a Green economizer, which is placed over the boiler setting in a by-pass between the boilers and the main breeching, the various economizers being a part of the induced-draft system. The different main parts of the latter have been designed to provide for draft for an eventual installation of six additional boilers, or twenty in all, sufficient for expansion for a considerable time.

The breechings, flues and connections have all been designed so that any part of the induced-draft system may be cut out of service without interfering with the operation of the other parts. The connection between each boiler furnace and the main breeching is fitted with a damper that is operated by hand from the firing floor in front of the boiler, so the amount of draft in each furnace can easily be regulated separately from the others. The by-pass in which the economizer for each battery of boilers is placed is connected at both ends with the main breeching. These two connections can be shut off either separately, or together by dampers, while a balanced damper in the main breeching at each economizer by-pass permits all the gases to be diverted through the latter if desired.

The main breeching over the boilers increases in size at a uniform rate to a cross-section of 10.5x12 ft. at the main flue. The latter is 14x14 ft. in cross-section for its full length from the junction of the breechings to the connections leading to the volutes of the fans in the house at the side of the building. The breechings and flue are suspended from the roof trusses of the building, while the economizers are carried by a steel frame in the boiler settings. In all cases ⅜-in. plates reinforced with channels and angles were used for the ducts carrying the waste gases from the furnaces to the fans. The limited head room over the boilers makes the installation of the ducts particularly difficult and necessitated the rearrangement of the piping connections by which the steam is led off from the boilers.

Engines which drive the fans are placed in the brick fan house outside of the power station building. Two smoke boxes, 11 ft. wide, are placed along the side of this house, one at each side of the end of the main flue leading from the boiler room and to these the volutes of the fans are connected. Dampers make it possible to cut each of these boxes out of service separately.

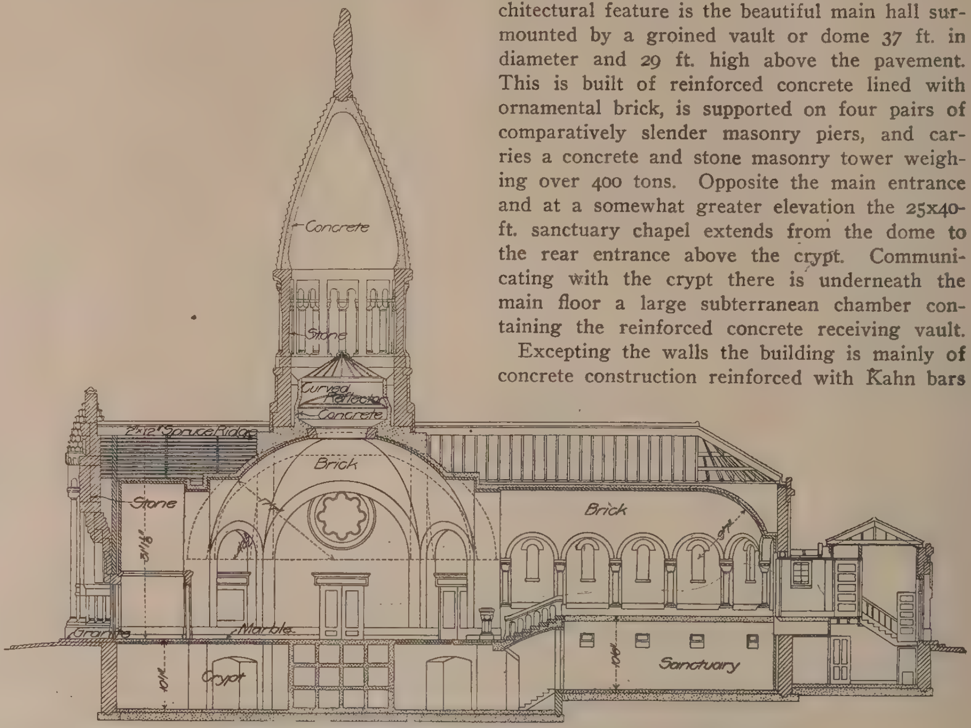
The two 20-ft. fans which furnish draft for the system are driven by two 12x30-in. Fulton Corliss engines in the fan house. Each fan is on the end of a shaft direct-connected to one of these engines, a double water-cooled bearing, each part of which is 48 in. long, being placed between the fan and the engine which drives it. The fans will normally be run at 80 r.p.m., but are designed to operate at from 70 to 150 r.p.m., depending on the amount of draft desired and on local conditions. Each fan is served by a self-supporting steel stack 10.5 ft. in diameter and 70 ft. high above the footing, which is integral with those of the fan house.

The additions to and the rearrangement of the power station are being made under the direction of Messrs. Lichter and Jens, consulting engineers, of St. Louis, who prepared the plans for and supervised the construction of the original station.

Combined Forced and Natural Draft Cooling Towers.

The Calvary Cemetery Mortuary Chapel.

The Mortuary Chapel now being completed in Calvary Cemetery, New York, at a total cost of about $200,000, is a 64x124 ft. structure about 32 ft. high to the top of the main roof and 92 ft. to the top of the tower, which is surmounted by a 5-ton stone figure 10 ft. high. The general elevation and the symmetrical plan are indicated by the vertical longitudinal sections and by the foundation diagram. The central architectural feature is the beautiful main hall surmounted by a groined vault or dome 37 ft. in diameter and 29 ft. high above the pavement. This is built of reinforced concrete lined with ornamental brick, is supported on four pairs of comparatively slender masonry piers, and carries a concrete and stone masonry tower weighing over 400 tons. Opposite the main entrance and at a somewhat greater elevation the 25x40-ft. sanctuary chapel extends from the dome to the rear entrance above the crypt. Communicating with the crypt there is underneath the main floor a large subterranean chamber containing the reinforced concrete receiving vault.

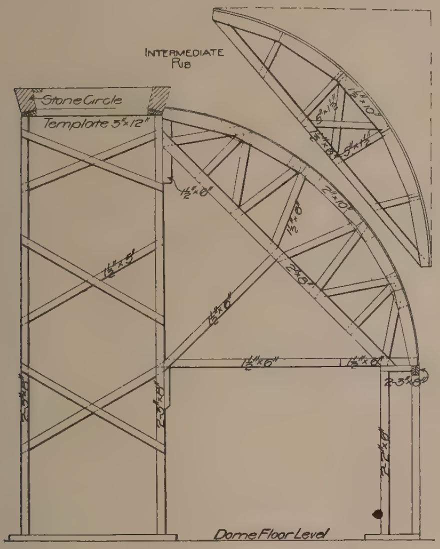


Section through Calvary Cemetery Mortuary Chapel.

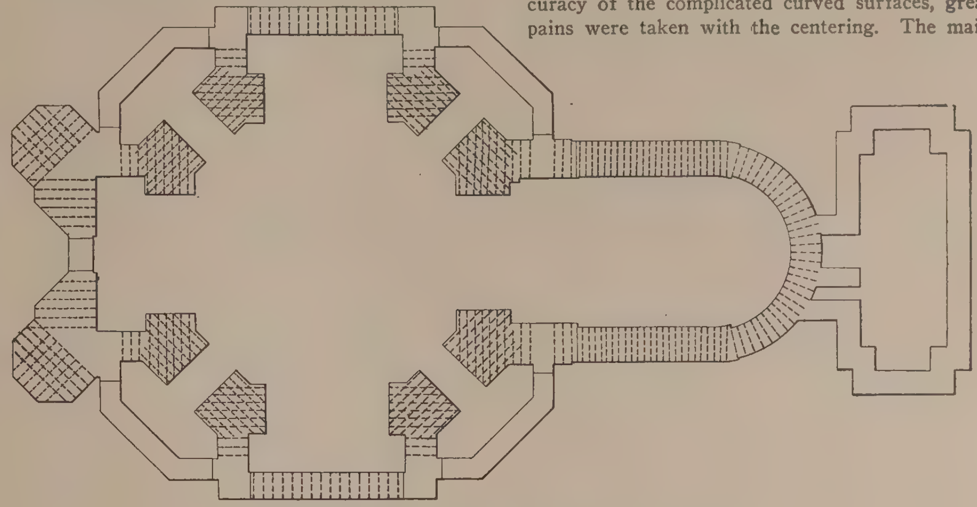
Excepting the walls the building is mainly of concrete construction reinforced with Kahn bars which make monolithic construction for all floors, roofs and dome. The walls are faced on the exterior with Indiana limestone above a granite base and the interior of the superstructure is faced with light brick and pink Minnesota sandstone. The main floor is paved with marble and that of the crypt and subterranean receiving chamber has a granolithic surface on a bed of cinder concrete. The doors and windows have stone lintels and the dome has at the crown a cut stone curb about 6 ft. in diameter, covered with a concave skylight of ornamental stained glass. Above the skylight there is a curved Frink reflector and a conical skylight roof at the base of the open colonade forming the lower part of the tower.

The vaulted roof over the chapel is a full centered arch surmounted by an exterior shell of timber framework covered with Celladon tile to give the Gothic appearance required for the exterior elevation. The foundations for the wall have continuous footings. Those for the 8 piers carrying the dome and tower are isolated. The dome piers are virtually segments of an octagonal ring taken at the vertices and their footings, 26 in. thick, are reinforced by cross layers of ¾-in. Kahn bars 12 in. apart on centers and 4 in. clear of the lower surface of the pier. The footings for the main walls are of the same thickness and are reinforced with single transverse bars of the same size. Under the crypt the footings are only 12 in. thick and are reinforced by 1½-in. transverse bars.

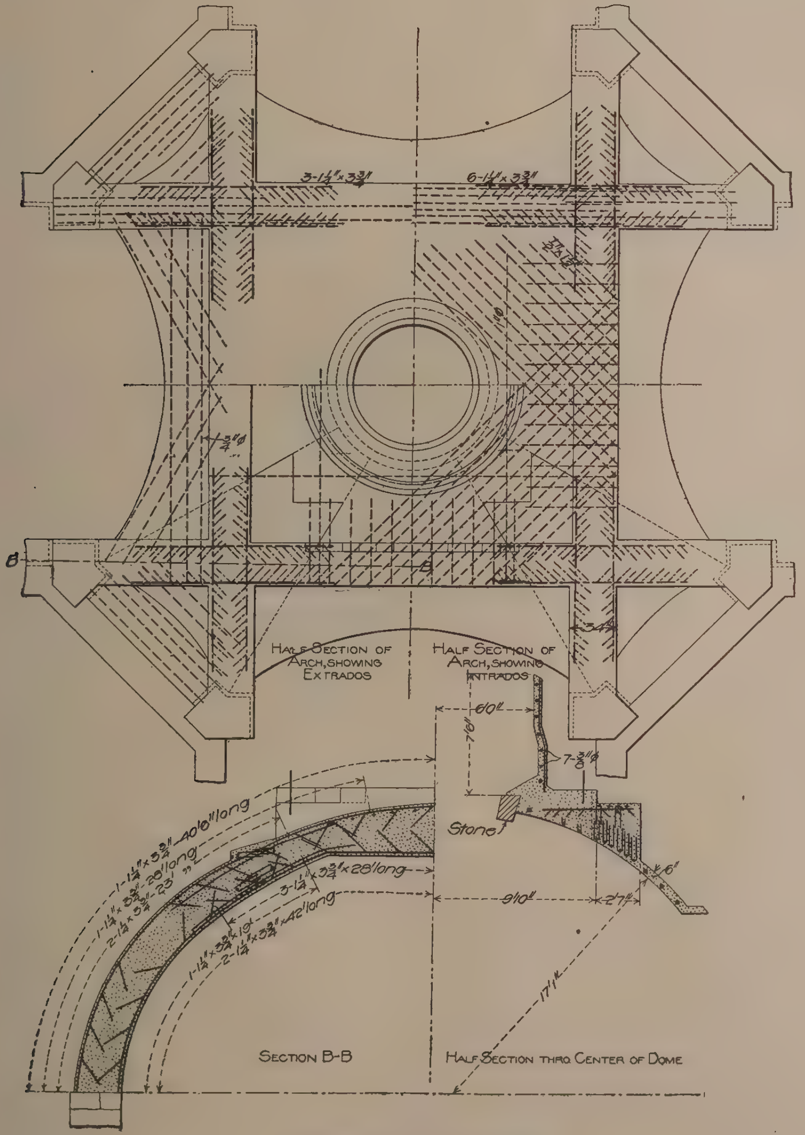
The reinforced concrete dome construction consists essentially of two pairs of arches at right angles to each other supported on piers. The tops of the piers are connected in pairs to arches of 9 ft. diameter of intrados and four 1x3-in. vertical anchor bars project above the top of each to secure the arches which were built separately after the completion of the piers. The thickness of the arches diminishes from 3 ft. on a horizontal line at the base to 2 ft. at the haunches where the thickness is a minimum at the intersection with the curves and horizontal intrados. They are reinforced with crossed layers of radial and horizontal bars near the intrados and extrados, as shown in the cross section details and have offset footings on the upper side to receive the stone masonry of the tower.



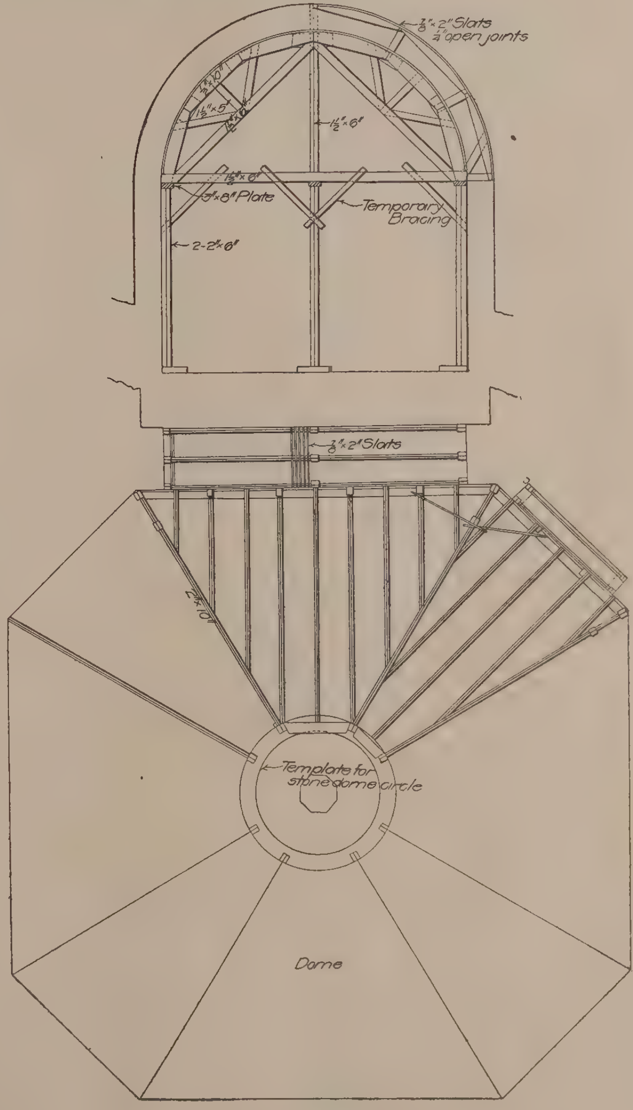
Falsework for Dome.



Plan of Chapel Showing Footing Reinforcement.



Reinforcement of the Dome.



Dome Falsework and Arch Centering.

The arches and groins of the 4-in. brick lining for the domes spring from so many points of intersection with the piers that the stability of the latter is very much decreased and the concentrated thrust would be so great that unless other supports were provided for them than is given by the skewbacks stability could not be secured. The brick lining is therefore connected by Morse anchors to the monolithic concrete dome above, from which it is actually suspended, although in construction the concrete dome was built on the brick arch.

In order to support the compound structure during erection and to provide for extreme accuracy of the complicated curved surfaces, great pains were taken with the centering. The main framework was built with 8 radial sections or sectors symmetrical about their center line and symmetrically arranged alternately in pairs corresponding with the long and short side of the octagon formed by the piers. Vertical posts consisting of pairs of 2x6-in. timbers were placed in the openings between the piers and in a circle under the crown of the arch. The former carried centering for the relieving arches and the latter carried segmental caps fitting the intrados of the dome. The ends of the caps and centers were connected by radial trussed ribs, like hip rafters, into which were framed transverse scarf boards at right angles to the separate elements of the dome arches and afforded support for the lagging which was very carefully cut and laid. The radial ribs were in reality wooden trusses with curved top chords made from the scarf boards and with 2x8-in. straight inclined bottom chords connected to them with 6x½-in. zig-zag web members spliced to both chords. The intermediate ribs were of similar construction but lighter and were proportioned to be supported at both ends without intermediate columns.

In order to make the courses of the brickwork exact and their alignment perfect, a full-sized quarter section model or negative of the intrados of the dome was made of plaster paris on which the courses of the brick work were marked and then transferred to the lagging. Every brick in the lining was applied to it and was cut and rubbed in two directions, namely, concave on the exposed face, concave on the lower bed, and convex on the upper bed so that the brick would be truly horizontal when laid. This work was done with great care and resulted in such precision that all of the lines are perfectly regular and no imperfections can be observed in the finished work. It is believed that this method of constructing brick work and suspending it from reinforced concrete above has never before been followed in this country, and so far as shown only once recorded abroad.

After the completion of the brick lining, forms were built above it and in them the concrete arches were made. After they were set the thin interior concrete walls for the sides of the lantern frame were built continuous with the dome concrete. Later the stone masonry walls for the base of the tower were laid in the usual manner on a footing provided by the offset upper surface of the dome, the concrete columns of the colonnade were set on it. Finally the reinforced concrete shell of the top of the tower 18½ ft. in diameter and 24 ft. high were built on it with sectional inside and outside wooden forms of ordinary construction. This concrete shell is reinforced by 12 horizontal rings of 1-in. round iron and by curved bars in vertical radial planes. The latter are made in two varying lengths. The base of the structure is reinforced by a ring of 6x6x⅜-in. angle iron secured to the sub-structure by eight 1¼-in. vertical anchor bolts through its horizontal flange. The vertical flange of this angle is bolted to the flattened lower ends of the radial reinforcement bars.

The stairway descending to the crypt is made with an inclined slab of solid reinforced concrete 12 in. in minimum thickness and provided on the upper surface with triangular offsets to engage the rectangular solid marble steps placed on it. The vaults in the crypt are contained in a monolithic rectangular concrete structure 14 ft. 4 in. wide, 17 ft. 8 in. long and 10 ft. high with horizontal and vertical partition slabs dividing it into 24 compartments each 30 in. square. The main walls are 8 in. thick and are reinforced with Kahn bars, all the partition slabs are reinforced with ⅜-in. round bars.

Mr. Raymond F. Almirall is the architect and P. J. Carlin Construction Co. is the general contractor. The reinforced concrete work was designed under the supervision of Mr. John Hawksworth, assistant engineer of the architect, by the Trussed Concrete Steel Co., Detroit, Mr. B. J. Greenhood, engineer, and was constructed by the Concrete Steel & Tile Construction Co., Detroit.

The New Water Supply of Mexico.

Heretofore the water supply for Mexico City has been of rather limited quantity and poor quality, but there is now in an advanced stage of construction a new system which will remove both objections. The new system comprises works for taking the water of several springs, a conduit to convey the water to the limits, receiving and distributing reservoirs, pumping plants, and a new system of distributing pipes in the city.

The new supply comes from four different groups of large springs at La Noria, Nativitas, Santa Cruz and San Luis, situated along the southwestern shore of Lake Xochimilco.

This lake is one of the ancient and historically interesting bodies of water in the valley of Mexico and lies on the southerly side of the city, the springs being from 20 to 30 kilometers from the reservoirs, which are located on high ground about 10 kilometers from the center of the city near to and westerly from Chapultepec.

The valley of Mexico, in which the city lies, is a great basin 7,000 ft. above sea level and entirely surrounded by high mountains. Of the latter, the Ajusco Mountains lie to the south of Lake Xochimilco, and their formation is so porous and their capacity for absorption so great that they appear to yield the underground flow from which the springs mentioned, as well as many others on the south side of the valley or basin, are supplied.

The water, as it comes from the springs, is as clear as crystal, and, analyses show it to be of a high degree of purity.

The four groups of springs yield a total of 700 gal. per second. The new works are designed to take and deliver 562.5 gal. per second, or 48,600,000 gal. per day. The population of Mexico City is about 400,000, from which it appears that the new supply provides about 125 gal. per capita.

To collect the water, reinforced concrete wells or caissons of rectangular and octagonal forms, with necessary partition walls, openings and valves, are sunk into and around the springs to depths of 20 to 30 ft. and with horizontal dimensions from 10 to 30 ft. The pumps are located in or over these wells.

To convey the water from the source of supply to the distributing reservoirs west of Chapultepec, a reinforced concrete conduit to have a total length of 30 kilometers (18.75 miles) is under construction. It runs across the level valley with a hydraulic grade of 0.0003. Its cross-section is oval, the maximum horizontal and vertical diameters being about 6.60 ft. The invert is flat, while the crown is, approximately, a parabola.

Along the conduit and spaced 333 meters apart, open concrete stand pipes about 20 ft. high and about 30 in. in diameter are built on the conduit with a manhole in the top of the latter on either side of the stand pipe, and a gate in the conduit between the standpipe and the manhole on the upstream side. The bases and tops of the stand pipes are handsomely finished in cut stone trimming.

The conduit will work under a normal head of 5 to 10 ft. Its shell above the invert is about 5 in. thick and the entire section is reinforced by expanded metal. The concrete is a 1:3:3 mixture, using Portland cement. Owing to the level country, the conduit has a covering of earth rarely exceeding 3 ft. except in approaching the reservoirs. It is laid but a few feet below and approximately parallel to the hydraulic grade.

The conduit terminates near and about 50 meters below the receiving and distributing reservoirs. As previously stated, the latter are situated westerly from the city and Chapultepec and adjacent to the west end of Chapultepec Park. The reservoirs are four in number and are at an elevation of 50 meters (164 ft.) above the streets of the city. Each one is circular and 100 meters in diameter and deep enough to hold 50,000 cubic meters, the combined capacity being equal to 30 hours' supply for the entire city.

For a depth of about 10 ft. the circular wall of the reservoirs is vertical, and from there to the bottom it slopes 1 on 1. The bottom is covered with a heavy floor of concrete. Each reservoir is covered with a heavy concrete roof of girders and slabs carried on concrete columns set on circles concentric around the central octagonal chamber about 18 ft. in diameter, which rises from the bottom to the roof of the reservoirs and terminates in an ornamental tower about 10 ft. in diameter above the roof.

This chamber has openings in each face inside the reservoir and is connected to a central gate chamber centrally located outside the reservoirs by a concrete tunnel having a diameter of 1.5 meters. The only office of this large central chamber in the center of each reservoir appears to be to permit the flow into and out of the reservoir, there being no gates or other means of control in it.

The roofs of the reservoirs are approximately level with the ground and, when finished, the reservoirs will be covered with earth and sodded, it being the intention to grade the reservoir site and convert it into an extension of Chapultepec Park.

Outside of and conveniently located near the reservoirs, one large controlling gate chamber, surmounted above ground by an ornamental tower, will be constructed. The tunnels above described will run from each reservoir to this chamber and deliver the water to it for distribution to the city.

All concrete in this construction is very heavily reinforced in all directions by Johnson bars, and the dimensions of the concrete are heavier than theoretical requirements dictate. The mixture is 1:3:3.

From the controlling gate chamber 60-in. cast-iron pipes will lead to the city and an entirely new network of pipes for the latter will be laid. This part of the work is not yet under way.

At each of the four groups of springs and wells the water must be lifted from 6 to 12 meters to get it into the conduit and give it necessary head. At the terminus of the conduit at the reservoirs, the lift will be 50 meters to get the water into the latter.

Single stage centrifugal pumps will be used for all this work, there being two 24-in. pumps at each of the four groups of wells, and one plant of three pumps at the reservoirs and having a capacity equal to the sum of the other four. These pumps are being constructed by I. P. Morris Co., Philadelphia, and will be run by electric power. The lift of 50 meters in single stage appears abnormal for this class of pump, but the data are taken from the official statement of the Water Works Board of Mexico City. Handsome stone buildings will house the pumps.

The work as a whole, which is being carried out in an admirable, permanent character, was designed and is being directed by Sr. Manuel Marroquin y Rivera, Chief Engineer to the Board.

IMPROVEMENTS IN MANILA HARBOR and the Pasig River, which were completed during the past year, cost $4,483,000. The work consisted in dredging channels and constructing breakwaters, and has resulted in making the harbor the safest in the Orient. Two covered steel and concrete docks, one measuring 600 by 70 ft. and the other 650 by 110 ft., are now being built. The mud dredged from the harbor and the river has been pumped to shore, and used in filling in along the water front, at the points recommended by Mr. D. H. Burnham, who visited the islands some years ago in order to plan broad improvements for Manila.

The Effect of Stress on the Corrosion of Iron.

It is doubtless known to a number of readers of this journal that Prof. William H. Walker and Mr. Colby Dill have been conducting some important investigations of the effect of stress on the corrosion of iron at the research laboratory of technical chemistry at the Massachusetts Institute of Technology. They presented a paper on the subject at the recent convention of the American Electrochemical Society, and were on the program for a similar paper before the convention of the American Society for Testing Materials, which was unfortunately read by title only. The substance of the information they have made public in these papers is an important contribution to our knowledge of the corrosion of iron. The authors consider that a definite relation exists between electromotive force and corrosion, and that the corrosion of iron in water depends essentially upon three factors, the electrolytic solution pressure of the iron, the electrolytic solution pressure of hydrogen, and the condition of surface of the iron or metal in contact with the iron, in so far as it affects the ease with which molecular hydrogen may be liberated on it. In an experiment, the last two factors may be held practically constant, and the electromotive force of the system made to depend for the greater part on the solution pressure of the iron. In the paper read before the American Electrochemical Society, the authors deal with a single one of the conditions which affect the electromotive force of iron, the effect of stress on the metal. The particular problem investigated was the determination of the sign and magnitude of the potential changes caused by straining a piece of iron, particularly below the elastic limit.

The magnitude of the increase of potential which one would expect to be produced can be easily computed on the assumption that the energy stored in the specimen below the elastic limit is available as potential. Some very pure Swedish charcoal iron was tested in the usual manner to determine the modulus of elasticity and the elastic limit. From these data the maximum amount of work which it is possible to do on 1 cu. in. of soft iron by stretching it to its elastic limit was calculated to be 5.16 in.-lb., which is equal to 5.83 joules. One cubic inch of soft iron weighs approximately 126 grams, the specific gravity being 1.7. The work done in joules per equivalent is therefore 1.30, and the change in elctromotive force which would be expected is 1.30 joules divided by 96,540 coulombs, or 0.0000134 volt. The magnitude of this change is, therefore, very small; its direction should be positive, because it is the manifestation of energy stored up in the metal.

The paper gives the results of experimental measurements by the Poggendorf method, using a cadmium cell as standard electromotive force. The specimens of iron tested were made from two lots of exceptionally pure Swedish charcoal iron. One lot was hard-drawn wire about 0.25 in. in diameter, and the other was bar iron about 0.5 in. in diameter. The bars were cut in lengths of about 18 in., and the central portion reduced in a lathe for a distance of about 1 in. until a zone of bright new metal was exposed. This band was smoothed in the lathe first with a file and finally with emery cloth. With the exception of a narrow zone about 0.25 in. wide at the middle, the bars were covered with several waterproof coatings of ordinary shellac to insulate the metal from the solution except at the desired point.

The cell consisted of a central tube about 3 in. long and 1 in. internal diameter, open at both ends. To this were bound by means of adhesive tape three smaller tubes of the same length closed at the bottom. The bottom of the central tube was closed by a rubber stopper, carefully cleaned, through which the iron part projected, so that the reduced portion came at the middle of the tube. The central tube containing the specimen was filled with ferrous sulphate and then the three outer tubes were filled, one with ferrous sulphate and two with potassium chloride. All four tubes were then connected by syphons. The syphon of the normal calomel electrode dipped into the last potassium chloride tube. These precautions were successful in preventing the diffusion of the $FeSo_4$ into the tube containing the normal electrode. The $FeSo_4$ solution was protected from air by a layer of paraffine oil carefully washed to remove traces of alkali or acid. The loads were applied with a testing machine operated by hand, as a power machine rendered potential measurements out of the question.

Tests were made with a large number of pieces of iron which were subjected to increasing stress up to the breaking point; a typical test was as follows: The stress was applied uniformly and gradually increased. The potential dropped simultaneously very slowly until at a stress of about 31,000 lb. per square inch of original section, the electromotive force had decreased 0.9 millivolts. When the machine was stopped at this point the beam of the machine sagged. When stress was again applied the potential rose suddenly 3.9 millivolts, and when the machine was stopped it dropped to its former value in 15 seconds, then sank more slowly to a minimum and then started slowly to rise again. When load was put on a second time the character of the change was similar to the first, there being first a sharp rise of electromotive force while the load was increasing, followed by an abrupt fall when the machine was stopped, then a slower fall, then a rise to a constant value. The magnitude of the sudden rise depended upon the rate at which the stress was applied. The final value after fracture was about 8 millivolts higher than the initial value.

The cause of the abnormal rise observed at high stresses was next investigated. Change in temperature suggested itself as the most probable cause, although iron under these conditions has shown itself to have a negative temperature coefficient. In order to duplicate as nearly as possible the thermal conditions which obtain in the iron electrode, a device was used whereby there was a continual flow of heat from the electrode to the solution. A hole about ⅛ in. in diameter was drilled throughout the length of one of the electrodes. The upper end was joined by a rubber tube to a reservoir directly above, holding about 2 liters. A copper-nickel thermopile was soldered to the surface of the electrode where it was in contact with the liquid. The copper, nickel and solder were insulated from the solution by means of shellac. Hot water was placed in the reservoir and allowed to flow down through the electrode, the rate being regulated by a screw pinchcock. The test showed that a rise in temperature produced a decided decrease of electromotive force. The experiment was repeated with additional precautions and entirely concordant results were obtained.

If the sudden rise in potential above the elastic limit is caused by temperature changes in the electrode, then, since iron has a negative temperature coefficient, the specimen must cool off as the breaking load is approached. Such a phenomenon, though highly improbable, is perhaps conceivable. Experiment showed, however, that there is a continuous rise in temperature from the elastic limit to the breaking load. The rise in temperature in this test was determined by means of the copper-nickel thermopile previously used.

An experiment was also made with normal ferric chloride as electrolyte to see if this change in potential below the elastic limit also occurred. The specimen was prepared in identically the same manner as the previous ones. Soon after immersion in ferric chloride it became coated with bubbles of hydrogen and the surface lost its metallic lustre and grew black. The potential rose rapidly during the first and second days. On the third and fourth days the potential was not measured. On the fifth day it had risen from 0.5279 to 0.6638 volt and was very constant. Scraping the specimen by means of a sharp wire produced a decrease of 0.0031 volt. On the sixth day the bar was pulled nearly to fracture. The potential decreased gradually 0.0006 volt from 1,200 to 32,600 lb. (yield point), per square inch. Here the potential rose as in all previous cases, but the subsequent behaviour was irregular. This experiment confirmed the previous ones with ferrous sulphate as electrolyte.

If there is any permanent difference in electromotive force between a strained and an unstrained piece of metal, it should be apparent in the case of hard-drawn wire when compared with the same wire annealed. A number of specimens of steel wire in its strained condition were obtained and portions of each specimen carefully annealed in a vacuum. In almost every case a difference of potential was observed between the annealed and unannealed specimen; but further investigation showed that as great, and frequently greater, differences existed between the different portions of the same wire, both in the strained and the annealed condition.

The results of the experiments are summarized by the authors of the paper as follows:

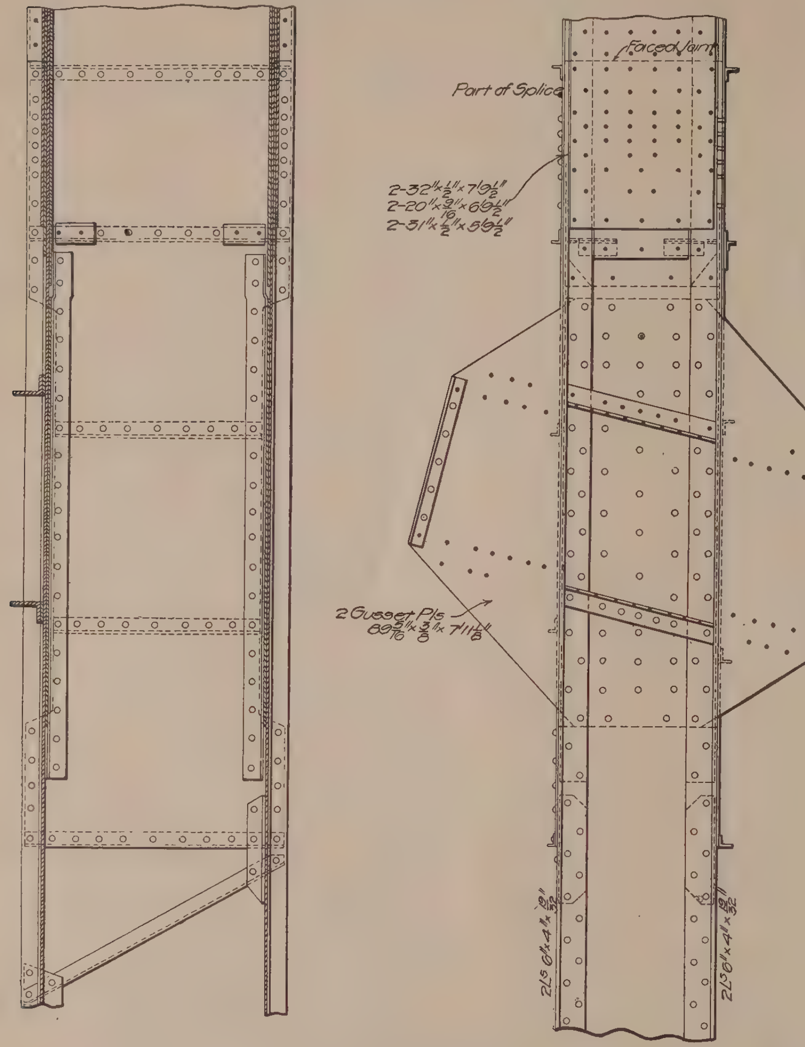
1. The magnitude of the potential changes suffered by soft iron when tested in a tension machine below the elastic limit is exceedingly small. In the majority of cases it was less than 0.0001 volt. The maximum change was 0.0004 volt.
2. The change, when great enough to be measured, was negative, *i. e.*, the strained metal had a slightly lower potential than the same metal unstrained.
3. Somewhere above the elastic limit the potential rises suddenly several hundredths of a volt. The magnitude of the increase depends on the rate of straining and ceases abruptly when the straining ceases.
4. Measurements on specimens under tortional stress give results similar to those obtained from tension tests.
5. Out of a considerable number of specimens strained to breaking, the potential of six reached a constant value shortly after fracture. The difference between the initial and final potentials varied from —0.00019 to +0.0077 volt, and the single potential of unstrained metal was found to be 0.156 volt.

Electrolysis of Pipes.

Electrolysis of pipes is receiving careful attention from the Metropolitan Water & Sewerage Board of Massachusetts, which states in its last annual report that the districts in which such damage occurs seem to be extending. In general, the injury has not proceeded so far as to require immediate repairs, but the examinations show that the corrosive action is advancing. The pipe lines crossing Chelsea Creek between Chelsea and East Boston were found to be particularly subject to disturbance, and at one point it was found that a hole had been eaten through a pipe. The examinations were made over the greater part of the system, records being made of the conditions for future use in caring for the mains. Efforts are being made to devise means of checking the action of the current, and as an experiment insulating joints have been inserted at several points in Cambridge.

The Quebec Bridge Superstructure Details; Part VI.

Diagonal Members.—The main diagonal in the panel at the river end of the anchor span is a pin-connected strut 273 ft. long on centers and weighs 268,000 lbs. It has a 38½x48½-in. rectangular cross section and was shipped and erected in four pieces with field riveted web and flange cover plate splices. At the lower end it has a cross section of 158.4 sq. in. gross, or 144.15 in. net proportioned for a total maximum statical stress of 1,312,000 lb. tension or 554,000 lb., compression and a wind stress of 1,990,000 lb. in either tension or compression. At the upper end the gross and net cross sections are respectively 226.9 and 208.9 sq. in. proportioned for maximum stresses, of 1,123,000 lb. static, and 2,350,000 lb. wind pressure in tension or 1,046,000 lb. static, and 2,350,000 lb. wind in compression.

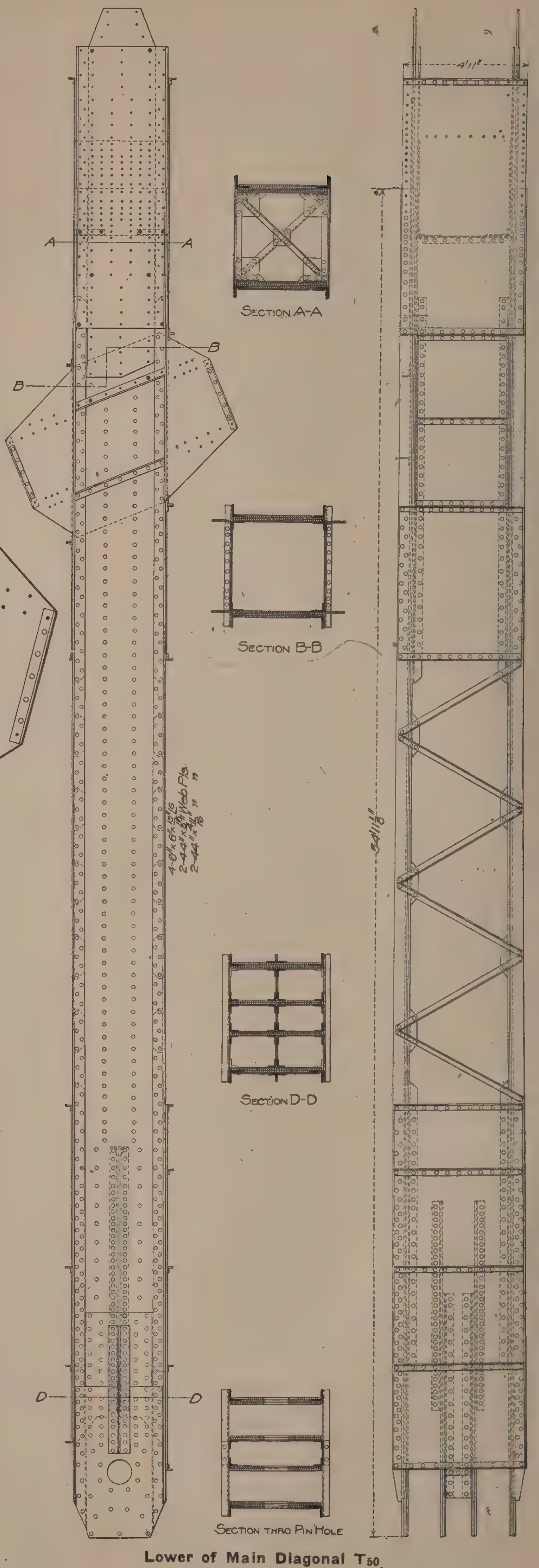


Part of Diagonal A S P₅.

The lower section of the member is about 55 ft. long over all and weighs 66,000 lb. It is made with two built channels having their flanges turned outward and latticed with 3x3-in. angles each of which has one rivet at each end through the channel flange and a second rivet through an auxiliary connection angle riveted to the inside of the channel web. The channels are each made with one ¾-in. and one 11/16-in. plates 44 in. wide and two 6x6x13/16-in. flange angles at the lower end. Additional pin bearing is providing by two intermediate webs about 13½ ft. long riveted between a center transverse diaphragm about 8½ ft. long and two flange cover plates.

For about 19½ ft. at the upper end of this section of the strut the cross section is closed by flange cover plates ½-in. thick, riveted on in three lengths and stiffened with transverse angles. The center lengths of these cover plates are 43 in. wide and 7 ft. 2½ in long and do not engage the flanges of the post channels, but are riveted on each long end to connection angles riveted in their turn to the webs of the post channels, thus connecting the latter rigidly together and still allowing transverse jaw plates to be riveted across their inner faces and project beyond both flanges to receive the field driven connections of the horizontal strut which intersects the diagonal strut at this point The splice between this section of the strut and the adjacent one is made with long multiple cover plates on the webs and single cover plates on the flanges which were shipped bolted in position with 10 shop rivets driven and finally connected with 1,100 steel field driven ⅞-in. rivets

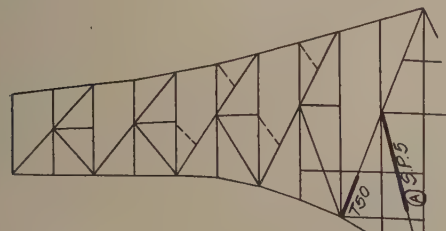
The next upper section of the strut is 87½ ft. long and weighs 80,000 lbs. with field riveted splices at both ends. Near the upper end a pair of 111x¾-in. wing plates 17 ft. 11 in. long are riveted to the backs of the channels and pro-



Lower of Main Diagonal T₅₀.

ject 67 in. beyond the flange on one side to form jaws for the field riveted connection of a sub-vertical, sub-diagonal and horizontal strut intersecting the main member at this point. The sub-vertical is continuous and passes between the webs of the diagonal strut to which it is field riveted. The outer edges of the plates are stiffened and connected by a diaphragm 7 ft. 9½ in. long field riveted in position after the connection has been made to the vertical post. During shipment the wing plates were additionally secured by thorough bracing and wedging with sub-units. The webs and flanges are stiffened and set in accurate position so as to maintain the rectangular cross section by interior X-braced frames or diagonals made up with angles such as are used for similar purposes in the outer vertical and sub-vertical posts.

The sub-diagonal strut in the river-end panel of the anchor arm has a riveted connection at both ends and is 195 ft. long over all with a total weight of 63,000 lbs. It is proportioned with a cross sectional area of 59.52 sq. in. designed for a total maximum static and wind pressure of 1,201,000 lb. It is made in three sections spliced together with inside and outside field riveted web cover plates. It is made with two built channels having a 32x9/16-in. web and two 6x4x19/32-in. flange angles turned outward and latticed with 3x3-in. angle bars each having two rivets at each end, one rivet being directly through connection plates interposed between the lattice bars and the channel flanges. A pair of 8x⅜-in. x 8-ft. gusset plates are riveted to the backs of the channel webs at the intersection of this strut with a horizontal strut and project beyond both flanges to form jaws field riveted to the latter. Between these jaws the channel webs of each column are connected by transverse diaphragms in the planes of the flange angles cut to clear the latter and the jaw plates, and riveted to short connection angles on the inner faces of the jaw plates. At this point the flanges of the channels are connected by a pair of 54-⅜-in. batten plates about 10½ ft. long stiffened by transverse angles and cut to clear the projecting wing plates to which, however, they are connected by inside angles riveted through them to the webs of the main channel.



Anchor Arm, Quebec Bridge.

(To be Continued.)

A Ganz Steam Motor Car is being tested by the Erie R. R. The car is divided into a motor compartment, a baggage-room, smoking-room, and general passenger compartment. It is 58 ft. long, weighs 45 tons and has seats for 50 passengers. The motive power consists of two compound inclosed steam motors of 60 h.-p. each, with cylinders 4.7 and 6.7 in. in diameter, and 5½ in. stroke. The steam generator is 42 in. in diameter and 5 ft. high, and produces superheated steam at 270 lb. pressure. Anthracite coal is used for fuel. The speed of the car is 40 miles per hour on level track. The car is being used in regular service out of the Jersey City terminal of the Erie. Cars of this general type have been used in Europe, and a western road sometime ago secured one for experimental purposes.

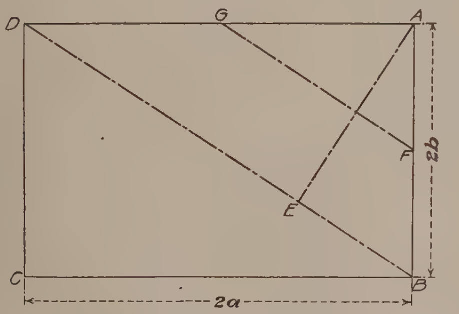
Notes on Reinforced Concrete Designing.

Accompanying the report of the Joint Committee on Reinforced Concrete, which was printed in this journal on July 27, were a number of appendices explaining the reasons for certain requirements suggested by the Committee. As these are of general interest in connection with the report, a number of them are reproduced below.

Bach's Slab Theory.—The first appendix was a discussion of Bach's theory of the resistance of flat slabs supported on all edges and uniformly loaded. It was written by Prof. A. C. Unwin and reads as follows:

The experiments of Professor Bach show that a flat square slab supported all round fractures along a diagonal, and the greatest stress is therefore on the diagonal section. It is the same apparently with rectangular slabs, though the evidence is not quite so clear. But if a diagonal fracture is assumed a very simple theory gives the stress.

Let the figure represent a rectangular slab with sides $2a$ and $2b$ in inches. Let the diagonal $B\ D = d$; the thickness of the slab $= h$; the perpendicular on the diagonal $A\ E = c$; draw $F\ G$ bisecting the sides and let p be the load per square inch. Consider the left-hand half of the rectangle. The total load on it is $2pab$ acting at the centre of gravity of $A\ B\ D$ or at $c/3$ from the diagonal. Whatever the distribution of the reactions of the supports, from symmetry, the reaction on $A\ B$ must act at the centre F of $A\ B$ and the reaction on $A\ D$ must act at the centre G of $A\ D$. Hence the resultant of the reactions on $A\ B$, $A\ D = 2pab$ must act at some point on the line $F\ G$ or at a distance $c/2$ from the diagonal. Hence the bending moment on the diagonal section is



$$M = 2pab\left(\frac{c}{2}-\frac{c}{3}\right)=\frac{pabc}{3},$$

the stress at the diagonal section is

$$f = \frac{6M}{dh^2} = 2p\,\frac{abc}{dh^2}.$$

$$\text{But } cd = 4ab,$$

$$d^2 = 4a^2 + 4b^2,$$

$$f = 2p\,\frac{a^2}{a^2+b^2}\,\frac{b^2}{h^2}.$$

The following form of the equation is convenient:

$$f=\tfrac{1}{2}\,\frac{\left(\frac{a}{b}\right)^2}{\left\{1+\left(\frac{a}{b}\right)^2\right\}\frac{a}{b}}\,\frac{W}{h^2}.$$

Where W is the total load on the slab:

a/b	1	1.5	2
f	$0.25\ W/h^2$	$0.23\ W/h^2$	$0.2W/h^2$

It would seem that if Bach's formula is to be used in calculating slabs, the reinforcing rods should be perpendicular to the diagonals of the rectangle.

Comparison of Slab Formulas.—The second appendix gives a comparison of the results obtained from the use of various rules for the strength of flat rectangular slabs supported on all edges and uniformly loaded. It was written by Mr. William Dunn and reads as follows:

The theories of Professor Grashof and of Professor Rankine assume that the maximum bending stress on the slab is at the centre, where there are two principal stresses on planes normal to each other, these planes coinciding with the major and minor axes of the slab.

The stress on the plane formed by the major axis of the slab (which is the greater of the two principal stresses) may be found in a simple manner as follows:

Let the length of the slab $= a$, and the breadth $= b$ (where a is equal to or greater than b).

Calculate the bending moment on the slab (disregarding the end supports) as a beam supported or fixed at the sides only, of a span b under the total load on the slab. Multiply this bending moment by the factor s in the following table, to allow for the effect of the end supports. The result is the actual bending moment on the long axis of the slab.

When	Grashof's and Rankine's Rule.		French Government Rule.	
$\frac{a}{b} =$	$s = \frac{a^4}{a^4+b^4}$	$r = \frac{b^4}{a^4+b^4}$	$s = \frac{1}{1+2\,b^4/a^4}$	$r = \frac{1}{1+2\,a^4/b^4}$
1.0	0.50	0.50	0.33	0.33
1.5	0.83	0.16	0.71	0.09
2.0	0.94	0.05	0.89	0.03

The stress on the section formed by the long axis of the slab is found in the usual way by equating this actual bending moment to the moment of resistance of that section.

Similarly the stress on the plane formed by the minor axis of the slab is found by assuming the slab supported or fixed at the ends (disregarding the effect of the side supports), calculating the bending moment as if the slab were a beam of span a under the total load on the slab. Reduce the bending moment so found by the factor r in the table above, and the result is the actual bending moment on the short axis of the slab.

The stress on the section formed by that axis is found as before by equating this moment to the moment of resistance of that section.

The reasoning by which we find the factors s and r is not entirely satisfactory, and other writers give other values. In the Instructions issued by the French Government to the bridge and highway engineers, with the report of the Ministerial Commission on Reinforced Concrete, the factors adopted give a greater importance to the effects of the third and fourth supports. The values of s and r, according to that report, are also given in the table above.

The maximum stresses on the sections as found by the foregoing rules when the slab is supported but not fixed all round are given in the table below, W being the total load uniformly distributed over the slab, h its thickness, and f the minimum stress due to bending.

When	Values of f according to Grashof and Rankine.		Values of f according to French Government Rule.	
$\frac{a}{b} =$	On L'g Axis.	On Sh't Axis.	On L'g Axis	On Sh't A
1.0	$0.375\ W/h^2$	$0.375\ W/h^2$	$0.250\ W/h^2$	0.250
1.5	$0.416\ W/h^2$	$0.183\ W/h^2$	$0.361\ W/h^2$	0.
2.0	$0.352\ W/h^2$	$0.088\ W/h^2$	$0.333\ W/h^2$	0

These results may be more readily
by the diagram.

It is implicitly assumed in the f
the strength to resist bending is th
directions, so that the reinforcem
and transverse should be of equa
same depth from the compresse

be placed parallel to both the ends and sides.

The stresses found by Bach's formula are also plotted on the diagram.

Moments of Inertia.—Prof. Unwin contributed the third appendix, on the moment of inertia of reinforced concrete, which reads as follows:

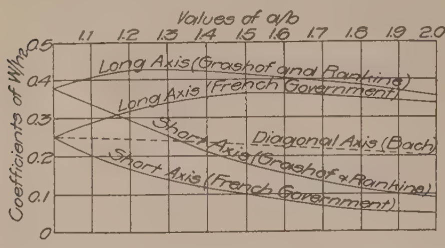


Diagram of Stresses.

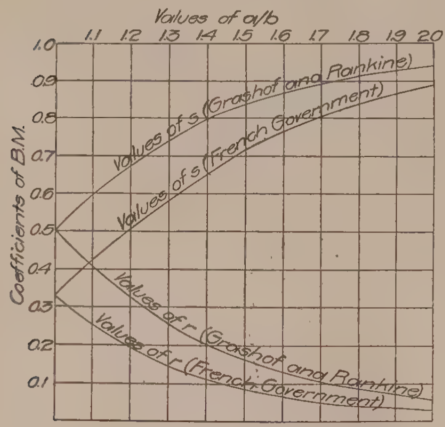


Diagram of Bending Moments.

If m is the ratio E_s/E_c of the coefficients of elasticity of steel and concrete, then an area A_t of steel is equivalent to resistance to mA_t of concrete. If A_c is the area of a section (including the area of the reinforcing bars), and a the area of the reinforcing bars, then the section is equivalent to a section of area $A = A_c + (m - 1)a$ of concrete only. This will be called the equivalent section.

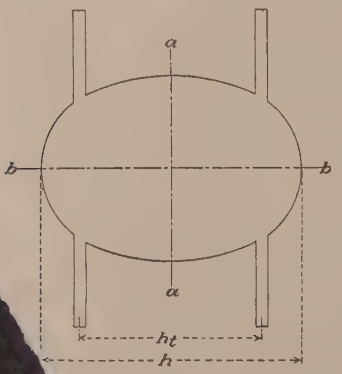
The moment of inertia of a section about its neutral axis can always be put in the form

$$I = nAh^2,$$

where h is the depth at right angles to the neu-axis and n is a constant depending on the form of the section. Thus for a rectangular section $I = {}^1/_{12}\,Ah^2$, and for a circular section $I = {}^1/_{16}\,Ah^2$.

In dealing with reinforced sections, it is convenient in many cases to express the moment of inertia in terms of the equivalent area. The equivalent area is found by adding to the actual area of the section portions of a total area $(m - 1)a$ at the same distance from the neutral axis as the reinforcing bars.

The figure shows a section for which $b\ b$ is the plane of bending, and $a\ a$ the neutral axis pass- [illegible]he centre of gravity of the section. [illegible]g bars are supposed symmetrical [illegible]xis. The projecting parts of total [illegible] are the concrete areas equivalent to the steel. If I_c is the moment of inertia of the section without reinforcing bars, the moment of inertia with reinforcing bars is



$$I = I_c + {}^1\!/_4\,(m - 1)ah_t^2.$$

Thus for a rectangular section

$$I = {}^1\!/_{12}A_ch^2 + {}^1\!/_4\,(m - 1)ah_t^2,$$

and the modulus of the section is

$$Z = {}^1\!/_6A_ch + {}^1\!/_2(m - 1)a\frac{h_t^2}{h}.$$

For a circular section

$$I = {}^1\!/_{16}A_ch^2 + {}^1\!/_4\,(m - 1)ah_t^2,$$

and the section modulus is

$$Z = {}^1\!/_8A_ch + {}^1\!/_2(m - 1)a\frac{h_t^2}{h}.$$

Example 1, Rectangular Section.—Let $m = 15$, $a = 0.01A_c$, and $h_t = 0.9h$. The equivalent area is $A = A_c + (m - 1)a = A_c + 14a = 1.14A_c$.

$$I = {}^1\!/_{12}\,A_ch^2 + \frac{14 \times 0.81}{4 \times 100}\,A_ch^2$$

$$= 0.1117A_ch^2$$

$$\text{But } A_c = 0.877A$$

$$I = 0.098Ah^2$$

$$Z = 0.196Ah.$$

In this case in the general expression $I = nAh^2$; $n = 0.098$ when A is the equivalent section.

Example 2, Circular Section.—Let $m = 15$, $a = 0.01A_c$, and $h_t = 0.8h$. The area of the equivalent section is $1.14A_c$, as before.

$$I = {}^1\!/_{16}A_ch^2 + {}^1\!/_4 \times 14 \times .01A_c \times .64h^2$$

$$= 0.0849\,A_ch^2.$$

$$\text{But } \boldsymbol{A_c = 0.877A.}$$

$$I = 0.0745Ah^2.$$

$$Z = 0.149Ah.$$

Example 3, Circular Section with Reinforcing Bars arranged in a Circle.—The reinforcing bars are nearly equivalent to a ring of steel of the same total area. Let $m = 15$, $a = 0.01A_c$, and let the diameter of the circle of reinforcing bars be $h_t = 0.8h$. The equivalent section is $A = 1.14A_c$, as before.

$$I = {}^1\!/_{16}\,A_ch^2 + \frac{m - 1}{8}\,ah_t^2.$$

$$= 0.0625\,A_ch^2 + 1.75 \times .01A \times .64h^2$$

$$= 0.0737A_ch^2$$

$$= 0.0646Ah^2$$

$$Z = 0.1292Ah.$$

It will be seen that if the value of n in the equation $I = nAh^2$ for simple circular and rectangular sections, and the reinforced sections are compared, the results are as follows:

	Simple Sections.	Reinforced Sections.
Case I....................	${}^1/_{12} = 0.0833$	0.098
Case II....................	${}^1/_{16} = 0.0625$	0.0745
Case III.................	${}^1/_{16} = 0.0625$	0.0646

The differences are not very great, so that while the value of n can always be found exactly when necessary for any proportion of reinforcement, there are cases such as that of columns where the value of n does not much affect the result, and where, from the nature of the calculation, great accuracy is impossible—for which a value of n can be assumed without any practically important error.

Ratio of Moduli of Elasticity.—The fourth appendix, on the ratio of the moduli of elasticity of steel and concrete, was written by Mr. Dunn, and reads as follows:

In the foregoing recommendations, as in the Prussian Government and various other foreign rules, the value of E_s/E_c for concrete of the kind usually employed, of hard stone or gravel mixed 1:2:4, is put at 15.

In reality, it varies with the age of the concrete, the proportions and nature of the materials, the stress at which it is taken, etc. As determined from tests of full-sized columns of concrete, with longitudinal reinforcement only and without transverse binding, it varied from a maximum of 10 at working loads to 15 to 21 at ultimate loads. For cinder concrete of 1:2:4 or 1:3:6 it varied from about 12 at working loads to 26 to 48 at ultimate loads.

This factor E_s/E_c is employed to determined the position of the neutral axis in beams, and it is found that while a variation between 10 and 15 makes no very great difference in the result, a value of 15 fixes the position of that axis nearest to the position found by experiment in singly reinforced beams. In fact, the formula given herein for the position of the neutral axis with $E_s/E_c = 15$ gives a result which agrees well with observed values in beam tests.

As mentioned above, it does not agree so well with the tests on columns: it gives too great an importance to the metal reinforcements, which becomes more noticeable when the percentage of reinforcement is considerable. But these tests were made on columns with longitudinal reinforcement only and without transverse binding, which latter adds greatly to the strength. This binding is explicitly required in the Prussian Government and other proposals where E_s/E_c is taken at 15; and that figure may be taken as making allowance in some degree for it.

We have no satisfactory determination of the increase in strength due to the transverse reinforcement, whether in single bindings or a continuous spiral.

In the Report of the French Commission on Reinforced Concrete, it is stated that where the concrete has spiral binding or transverse or oblique reinforcements, so disposed as to resist swelling under thrust, the safe loads may be increased in some measure, but not in any case to more than ${}^6/_{10}$ of the crushing strength as determined from tests on cubes of 20 cm. sides at the age of 90 days.

In the explanatory circular accompanying that report the value to be given to E_s/E_c is discussed, and its theoretic value is put at about 10. It is stated, however, that it is preferable to regard that coefficient as the result of experiences on pieces with longitudinal and transverse reinforcements, and not as representing the ratios found from concrete and metal separately. It is to be taken as varying from 8 when the longitudinal reinforcements have a diameter equal to one-tenth of the least dimension of the piece and the bindings are spaced at a distance equal to that least dimension in the direction of the length; up to 15 when the longitudinal bars are one-twentieth of the least dimension and the bindings are spaced one-third of that distance apart. In both cases the bindings are to be near the outer face of the concrete.

Again, it is stated in the circular that it is desirable to encourage the proper use of the metal in both longitudinal and transverse directions. While an exact determination of the increase of strength due to the transverse reinforcement would be difficult, the investigations of the Commission enable it to be admitted, in default of something better, that it is found by multiplying the resistance to crushing of the concrete by a coefficient.

$$1 + m^1V^1/V,$$

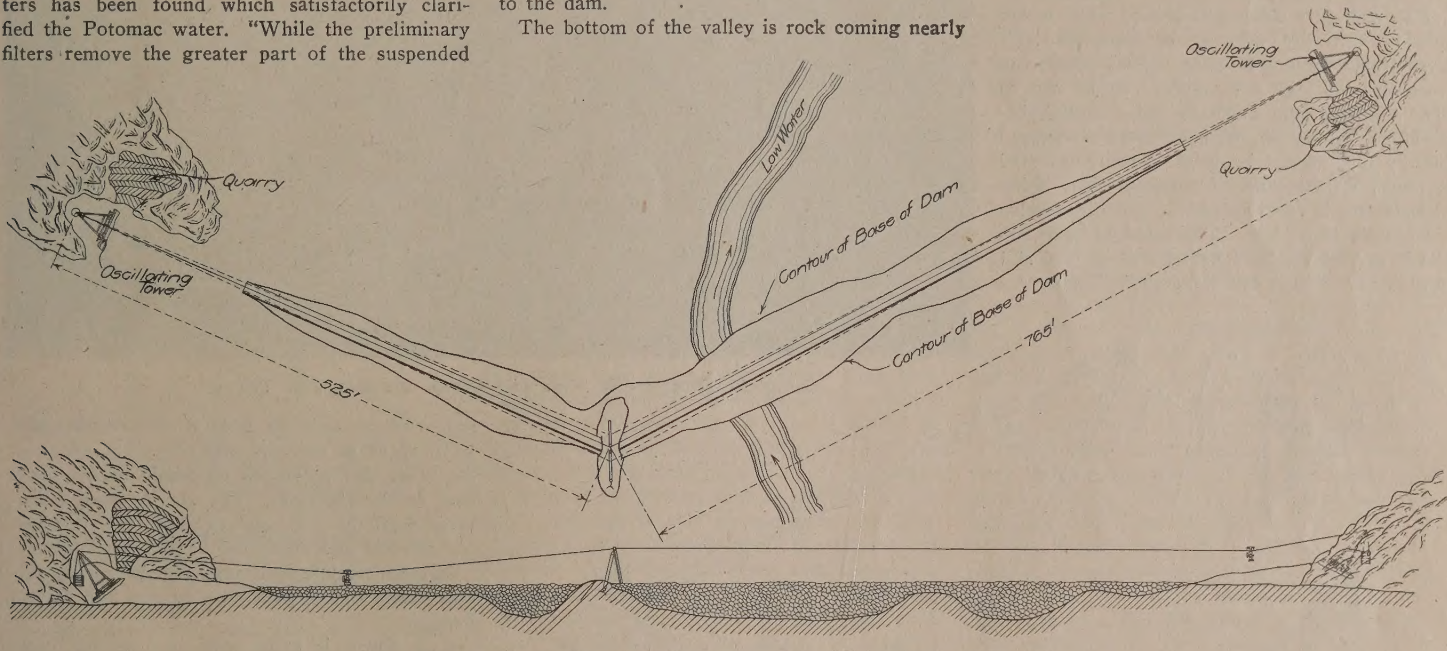
V^1 being the volume of the transverse or oblique reinforcement, and V the volume of the concrete, m^1 being a variable coefficient depending on the efficiency of the union between the longitudinal bars. When the union is made by bindings in the usual way, m^1 varies from 8 when these bindings are spaced at a distance equal to the least transverse dimension of the piece to 15 when spaced at one-third that dimension.

When spiral binding is used m^1 varies from 15 to 32, the lower value being taken when the pitch of the spiral is two-fifths of the least transverse dimension, and the higher when the spacing is one-fifth of the dimension under a pressure of 50 kilos per square centimeter, or one-eighth of the dimension under a pressure of 100 kilos per square centimeter.

In no case should the working stress exceed six-tenths of the resistance to crushing as determined on cubes, as before mentioned.

Filtration Experiments at Washington.

The water purification experiments conducted under the direction of Major Spencer Cosby, Corps of Engineers, at Washington, were described at some length in this journal on May 25. They were made with three systems of treatment: A, a rapid sand filter and a slow sand filter; B, a Maignen scrubber and a slow sand filter; C, a settling and coagulating basin and a slow sand filter. In a report recently submitted to the Chief of Engineers by Major Cosby, it is stated that no combination of filters has been found which satisfactorily clarified the Potomac water. "While the preliminary filters remove the greater part of the suspended matter," he says, "and leave comparatively very little work for the slow sand filters to do, the grains of mud remaining in the water after the first filtration are so exceedingly small that they are not entirely removed by either a second or third filtration. In order to arrest these fine grains a layer of very finely powdered animal charcoal was applied to the surface of the slow sand filter of system B, but without any apparent success. The results from system C (the modified slow sand system) show, however, that it has so far been successful in keeping the water free at all times of turbidity by the occasional use of a coagulant and subsequent filtration. The experience thus far gained with this system indicates that it will be found to be the most practical method of clarifying the Potomac water at a reasonable cost, and at the same time utilizing the plant already installed. The bulk of the mud is not only removed by coagulation, but the small amount which remains is so changed in character that it can readily be removed by filtration. It also tends to form a sticky gelatinous mass on the top of the filter sand which assists in arresting particles of mud in the applied water for several days after the use of the coagulant has ceased." The experiments were undertaken in order to find the best means of removing the turbidity which sometimes occurs in times of flood.



Two Span Continuous Cable Crane, Nevada-California Dam,

The Construction of the Dam of the Nevada-California Power Co.

A dam about 1,300 ft. long and 75 ft. high is now under construction by the Nevada-California Power Co., across a narrow valley at a high altitude in the Sierra Nevada Mountains to impound water for supply and power development purposes. It was at first intended to build the dam of stone masonry laid in cement mortar or of concrete, and derricks and other appliances were shipped to Laws, Cal., on a narrow-gauge railway at a cost of 6 cents per pound freight from New York. From this point to the dam site, a distance of about 18 miles, it is necessary to haul all materials by wagon over a very steep, rough road at a cost of $80 per ton. This made the transportation of appliances and materials so costly that the design and method of construction had to be radically changed, and it is now proposed to build the dam entirely of loose rock puddled on the upper side, quarrying it at the site, and handle it by a cable crane system so light that its installation will require the minimum amount of materials to be transported from Laws to the dam.

The bottom of the valley is rock coming nearly or quite to the surface of the ground and rising near the middle of a valley in a ridge extending a little above the crest of the dam and forming a center abutment where the two straight portions of the dam intersect in an angle of about 140 deg. pointing up-stream as indicated in the general plan. On one side of the ridge the dam is about 350 ft. long, 55 ft. wide at the base and 30 ft. high. On the other side it is about 570 ft. long, 75 ft. wide at the base and 35 ft. in maximum height. The crest of the dam has a uniform width of 10 ft.

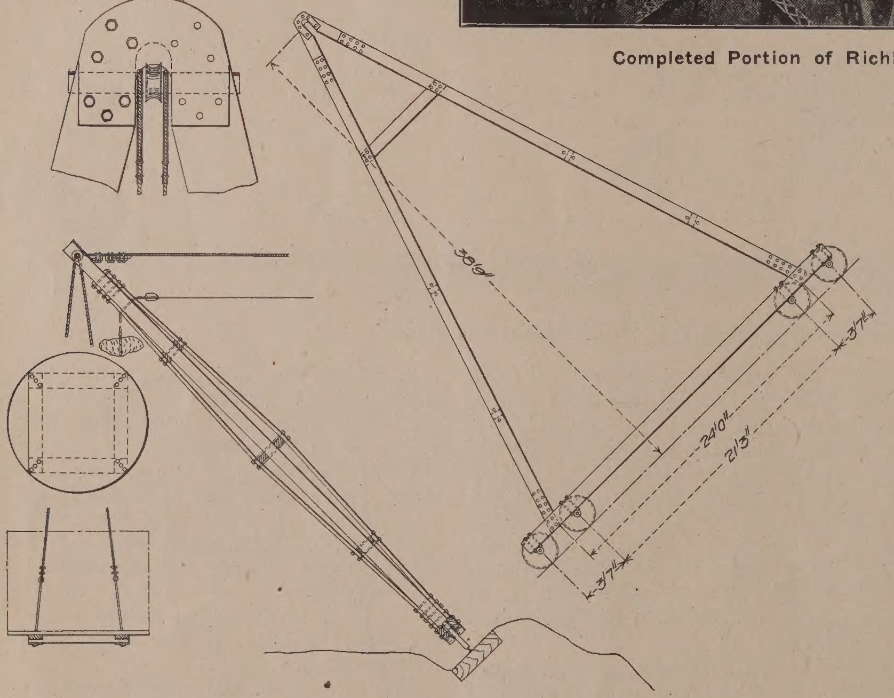
The axes of the dam produced one or two hundred ft. beyond each end, intersect the faces of steep cliffs of solid rock rising many hundred feet above the crest of the dam. In these cliffs natural recesses occur at the right places to enable them to be utilized for the location of the end towers for the cable crane in positions thoroughly protected from operations in the quarries which will be commenced on the face of the cliff between the recesses and the ends of the dam. In these quarries, carried up to any convenient height above the cable crane, the rock will fall by gravity as blasted to the foot of the cable crane and can be delivered and deposited by it in the required position in the dam, thus obviating the use of all derricks or tracks and reducing the handling of material.

The two-span, cable crane system has been designed and built by the Balanced Cable Crane Co., New York, Mr. W. F. Brothers, president. The plant installed has a single Victor cable 2 in. in diameter with a capacity for a 5-ton load and a breaking strength of 200 tons. It is 1,290 ft. long, and is attached at both ends to oscillating traveling shears which gives it sufficient transverse movement to enable it to command the full width of the dam. It is divided into spans of 525 anad 765 ft. by a telescopic pivoted strut set on the rock at the intersection of the two sides of the dam. The strut bisects the angle between the axes of the dam and is inclined from the vertical toward the stress in the cable and is secured by a tackle anchored to the solid rock. Each tower is inclined away from the center of the dam so as to exert a constant tension on the cable, and has suspended from its apex a 40-ton counterweight exerting a constant tension on the cable and capable of sufficient vertical motion to allow the top of the tower to revolve through an arc sufficient to permit the adjustment of the angle to the position of the load and maintain a constant tension in the cable. These conditions being similar on both sides of the center support the whole system remains in equilibrium while both spans act independently. Both towers are seated on single-track rails perpendicular to the axis of the cable, and when it is necessary to move either one the other can be adjusted until the counterweight is seated with bearing on the solid ground. In the 526-ft. span the deflection due to weight of cable and to maximum load are respectively 2.41 and 23.26 ft. In the 765-ft. span the corresponding deflection was 6.19 and 39.54 ft., the maximum cable tension in both spans being 42 tons.

Each of the duplicate towers consists of a triangular wooden frame about 39 ft. high and 31 ft. wide over all, made with single 10x10-in. legs trussed with 10x3-in. planks and with a single 6x12-in. sill. There are four 24-in. wheels with their single flanges set alternately on opposite sides of the rail. The tops of the posts are bolted together through a pair of ⅜-in. plates and have a horizontal transverse pin receiving the clamped loop-eye of the main cable and the bights of two suspender ropes for the counterweight platform. The platform, 10 ft. in diameter, is made of 3-in. planks and is loaded with 40 tons of rubble masonry or concrete. Below it a cushion 2 or 3 ft. deep of soft earth is provided where it can be lowered down to bearing in case it is necessary to temporarily seat it on the ground. The single 80-lb. rails are spiked to short ties perpendicular to the main inclination of the tower.

The hoist carriage is suspended from two deeply grooved wheels with teeth cut in their flanges which mesh with a pinion on a horizontal axle, which, at its outer end, is propelled, released or gripped by the action of a single lever controlling it through friction clutches which directly engage the motor. The pinion can be reversed to propel the carriage in either direction and it is traversed, held stationary or allowed to coast by different positions of the same lever. Each of the two hoist ropes is double with its bight wound on a separate drum and the loose ends provided with clevises for attachment to the bucket or skip. Each drum is supported on four sets of ball-bearings with steel racers about 15 in. in diameter and ⅝-in. hardened steel balls. The hoisting apparatus is controlled by a single lever and clutches much like those for traversing the carriage, and it is so arranged that either drum can be disengaged and run, stopped or overhauled independently of the other, thus enabling the operator to revolve a skip about its horizontal axis to dump it or to operate a clam-shell bucket. The motor shaft is vertically midway between the belt-driven traversing and hoisting pulleys, which run constantly. The 40 h.-p. motor can be run in either direction and normally drive both hoisting and traversing shafts, but can be disengaged instantly from either in order to concentrate all the power on the other if required. The hoisting apparatus is rated at 40 h.-p. and can develop a maximum of 60 h.-p. The speed of operation is 80 ft. per minute for hoisting a 5-ton load and 1,500 ft. per minute for traversing. There is a duplicate carriage on each cable and each carriage has a suspended enclosed cab for the operator who is in entire control of hoisting, traversing and dumping. A 50 h.-p. General Electric dynamo, driven by a 75 h.-p. engine, furnishes the current which is transmitted through a trolley wire attached to both towers and provided at each end with an adjustable counterweight.

The surface of the rock is being stripped and a concrete foundation prepared for the sluice pipe intended to provide for the dry weather flow through the dam while the latter is under construction. It is expected that the quarry blasts will deliver the broken rocks so close to the cable crane at both ends of the dam that a large proportion of it can be chained in massive pieces and delivered to the required position at an average cost of about 25 cents per yard. This is on the assumption that 1,000 cu. yds. per day can be handled by a small force of laborers at 50 cents each per hour.



Oscillating Tower, Nevada-California Dam.

Messrs. Manifold & Poole, Los Angeles, Cal., are the engineers of the power plant.

The Specific Heat of Superheated Steam has been under investigation for some six years by Mr. A. R. Dodge, and at the recent meeting of the American Society of Mechanical Engineers he presented a paper describing his methods and summarizing his conclusions. He believes that at constant pressure the specific heat is constant for all ranges of temperature investigated, although it increases with increasing pressure according to the equation $c = 0.4754 + 0.00031\ p$.

Richland Creek Viaduct, Indianapolis Southern Railway.

A single-track viaduct of the Indianapolis Southern Ry. crosses Richland Creek, 7 miles from Bloomfield, Ind., with a steel superstructure 132 ft. in maximum height and 2,215 ft. in length. It has alternate 40 and 75-ft. plate girder spans, supported on eighteen 40-ft. towers, besides two 50-ft. spans at one end and two 60-ft. spans at the other end supported on single rocker bents next to the abutment. The girders have a uniform depth of 7 ft. and are spaced 8 ft. apart transversely and connected by the usual vertical transverse sway-brace frames and by zig-zag lateral angles in the planes of the top and bottom flanges.

Completed Portion of Richland Creek Viaduct and Traveler.

Each tower is made with two vertical bents 40 ft. apart longitudinally, each vertical bent being composed of two columns battered 2¼ :20 with open rectangular cross-sections made with two built channels composed of pairs of 4x4-in. angles and 21-in. web plates latticed on flanges. The thicknesses of the angles and plates are the same in each column section and vary from ⅜ to ⅝ in.

The longitudinal faces of the columns are divided into two, three or four panels and the transverse faces into four, five or six panels, according to height, and both longitudinal and transverse panels are X-braced with struts, all of which are made with pairs of 10-in., 20-lb. channels latticed, with double 2½x⅜-in. bars. There are no horizontal struts except at the top and bottom of the transverse faces of the columns and where, both bent of the tower being of the same height, the longitudinal struts connecting the feet of the post in both bents happen to be horizontal. All bottom longitudinal and transverse struts are supported at their center point by suspenders made with four angles reaching to the intersection of the diagonals in the panel above.

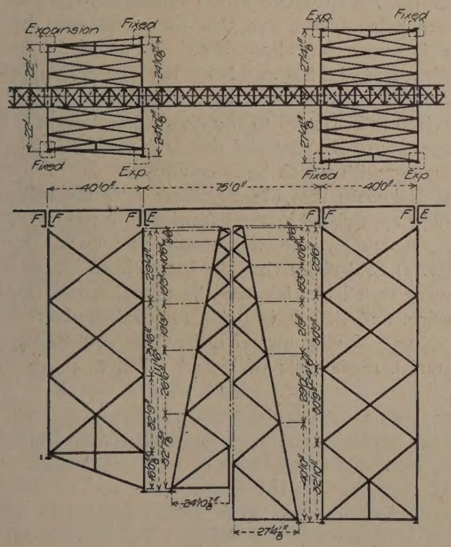
The feet of opposite diagonal columns in each tower have expansion bearings with a ⅜-in. phosphor bronze plate sliding on a ⅞-in. steel masonry plate. The other columns have fixed bearings on 1¼-in. steel masonry plates. There are no transverse girders and the tops of the columns have horizontal cap plates directly receiving the ends of the longitudinal girders with expansion bearings at one end of each 75-ft. span. Cross ties are laid on the top flanges of the longitudinal girders to receive a track on a tangent and at 5 per cent. grade.

The structure was designed for Cooper's E-50 live load and unit stresses of 15,000 lb. and 19,000 lb. were allowed in tension reduced by the impact formula, $I = LL \times \frac{LL}{DL + LL}$.

The maximum unit stresses in compression were

$$\frac{15000}{1 + \frac{l^2}{13500r^2}} \quad \text{and} \quad \frac{19000}{1 + \frac{l^2}{13500r^2}} \quad \text{and}$$

a maximum of 9,000 lb. per square inch was allowed for shear. Unit values for shop-driven rivets were 11,000 lb. shear and 22,000 lb. bearing, and for field-driven rivets 8,800 lb. shear and 17,600-lb. bearing.



Towers and Connecting Span.

The superstructure contains 2,017 tons of steel fabricated at he Detroit plant of the American Bridge Co., of New York and erected, as described in The Enginering Record of February 23, 1907, by the Strobel Steel Construction Co., of Chicago. The design and construction was under the direction of Mr. R. E. Gaut, enginer of bridges of the Illinois Central R. R. Co.

A New Use of Tar.

A new use of tar is reported from East Palestine, Ohio, where a mason erecting a number of brick houses ran short of black mortar color and was unable to get a new supply in the time at his disposal. Accordingly he tried a little partially refined tar, and had no difficulty in getting the right color for the mortar for pointing and beading between the bricks. Fearing defects from this material, the mason watched the houses very carefully, and recently reported that after a lapse of several years he found the color as strong as ever.

WATER-POWER DEVELOPMENTS are being investigated in the southern part of Sweden for furnishing power to operate the State Railways in that part of the country. Estimates have been made which show that five waterfalls which can be developed for about $16,000,000, will furnish sufficient power.

Special Methods of Shaft-Sinking.

A paper read before the Engineering Conference of the Institution of Civil Engineers by Henry Louis.

Under the head of special methods of shaft-sinking, I propose to consider the methods that have to be resorted to when ordinary methods cannot be applied economically on account of an excessive influx of water. Such complications are mostly confined to coal-mining, and arise but rarely in metalliferous-mining, mineral deposits usually lying in the older, less heavily-watered rocks. Year by year, as the coal-seams available within the area of the exposed coal-fields are more extensively drawn upon, it becomes increasingly necessary to sink down to the so-called concealed coal measures, overlain by more recent formations, many of which carry vast quantities of water.

It is a question what quantity of water may be regarded as necessarily involving the use of special methods. The largest amount that I know to have been sunk through successfully was at Horden Colliery, where 9,250 imp. gal. of water per minute at a depth of 540 ft. were dealt with. But in many cases far less than this amount has proved an insuperable obstacle to sinking in an ordinary way. It is possible that in the future ordinary methods of sinking may be found applicable in cases in which they have heretofore been considered impracticable, by the use of suspended tubbings, as introduced by Messrs. Haniel and Lueg of Dusseldorf, a system which has never been tried in this country, although it has been used with much success in several deep sinkings on the Continent.

The special methods to be used will differ, according as the water-bearing strata to be traversed are firm or running ground. In the former case the Kind-Chaudron method is mostly resorted to. This system was first employed about the year 1854, since when about eighty shafts have been sunk by it, of which five have been in this country. A pair of shafts were put down by it in 1876 at the Cannock and Huntington Colliery in Staffordshire, but the operation failed through the breaking away of the ground at the moss-box in each shaft. Sinkings by this method were also commenced, almost simultaneously, in 1877 at the Whitburn Collieries, and were completely successful. Finally, the shaft of the Consolidated Kent Collieries Corporation, Limited, was sunk by this method at Dover in 1904-5. A depth of 1,095 ft. had been reached by ordinary methods of sinking, and there still remained some 90 ft. of Oolitic and Liassic rocks to sink through in order to reach the coal-measures. This was done by the Kind-Chaudron method, and a column of tubbing 1,120 ft. high was put in. The modern continental practice in such cases is not to put in a column of tubbing the full depth of the shaft, but to lower a length of tubbing corresponding only with the depth bored out, and fitted with a false cover as well as a false bottom, the economy being obvious.

Mr. Riemer, one of the leading German authorities on sinking, considers that it will not be possible to execute sinkings by the ordinary Kind-Chaudron method at depths exceeding 2,000 ft. because it would be impracticable to handle cylinders thick enough to withstand the pressure corresponding to that depth, the limit of thickness according to him being about 5 in. Various proposals for modifying the method, so as to render it available at these great depths, have been put forward by Mr. Riemer and Mr. Tomson, but have not yet been put into practice.

When the ground to be sunk through is running as well as water-bearing, the Kind-Chaudron method is no longer applicable. In such cases three groups of methods are recognized, namely (1) driving down annular sheet-piling, (2) forcing down continuous cylinders of brickwork or iron, and (3) the Poetsch freezing process. Annular piling is the oldest method, and is only applicable when the stratum of soft ground to be passed through is not very thick, and is comparatively near the surface. As the piling is usually in lengths of 15 ft. to 20 ft., and as each new ring of piles has to be driven inside the one above, it follows that the original diameter must be taken very much greater than the finished diameter of the shaft. The method is not often seen now, but a good modern example may be quoted, namely, the recent sinking (September, 1906) at Bowburn Colliery, Co. Durham.

It may be here suggested that ferro-concrete sheet-piling might be used. Piles of this material, 60 ft. in length, have been driven, and it has also been found practicable to add fresh lengths to the top of piles already driven, and thus to get down to almost any desired depth. Furthermore, the adjacent sides of ferro-concrete piles can be grooved, and when the piles are down, cement can be run into the spaces thus left, making the piling practically water-tight. As far as I know, this system has never been tried for shaft-sinking, but there appears to be no reason why it should not be perfectly successful.

The method of sinking continuous cylinders, and excavating the ground from inside them, has only been used in this country for moderate depths. There have been several good examples of shallow sinkings by it in Scotland, where sand, mud and silt on the seashore have been sunk through for depths of about 100 ft., the sinkings at Musselburgh (1901-2), and at Bridgeness (1878), being cases in point. In a more recent sinking of a pair of shafts at Ardeer (1905), about 80 ft. of silt had to be traversed, and a similar process was adopted, using, however, the somewhat antiquated device of the air-lock (first used by Triger in 1839) for the actual excavation, the men working in compressed air at the bottom of the cylinder. Modern practice seems generally to favor the use of some form of dredge for excavating the material from the inside of the cylinder.

This principle has reached its greatest development in Germany, where it has been extensively used for sinking down to firm ground through the overlying thick masses of superficial deposits. The modern practice consists in sinking a cylinder, usually of masonry, built upon a massive iron cutting shoe, until skin-friction prevents its further descent. Tie-bolts, attached to the cutting shoe, are built into the brickwork, and by means of these a heavy iron anchor-ring is secured to the upper part of the shaft. A cylinder of iron is then built up beside the first cylinder, and is forced down by powerful hydraulic presses, which work against the anchor-ring. The ground inside the cylinder is excavated by some form of dredge, grab-dredges, bucket-dredges, bag-dredges, and the air lift all having been made use of.

The freezing method was devised by F. H. Poetsch in the year 1883. Although originally intended for sinking through wet quicksands, it has also been applied to sinking in solid but broken strata, carrying much water. Although it had been used repeatedly with success in both France and Germany, the first applications of this method in Britain have been comparatively recent, and have all been confined to the Durham coal-field. The first example was the sinking of a pair of shafts at Washington in 1902.

Freezing was subsequently employed in two very difficult sinkings at Eastington and at Dawdon. At the former place the operation was a failure; the yellow sands here are exceptionally thick, namely, over 100 ft.; they lie at 465 ft. beneath the surface; the water is brackish, and the water-level is affected by tidal movements. At the neighboring colliery of Dawdon, however, the operations, commenced about the same time, were perfectly successful.

Another very interesting sinking by this method is now in progress at the famous Wearmouth Collieries, originally sunk in 1826. It is now proposed to sink a new shaft close to the site of the old ones. If this were done by the ordinary method, the necessary pumping would relieve the tubbing in the old shafts of the hydrostatic pressure under which it now stands, and when this pressure comes on again, which it would do as soon as the new shaft were tubbed off, the old tubbing would in all probability be destroyed. Hence it has been decided to use the freezing method, which does away with the need of pumping, and therefore does not affect the hydrostatic level of the district.

Finally, reference may be made to a method that has only been used once or twice in the North of France, namely, the forcing of cement slurry through boreholes into soft, fissured strata, and thus forming a wall of concrete within which sinking can be performed; this method was used successfully a few years ago at the Lens Collieries.

Book Notes.

An attractive 26-page pamphlet has just been issued giving a brief description of the general design and construction of the Quebec bridge now under construction across the St. Lawrence River at an estimated cost of about $4,000,000. It gives a very concise resumé of the principal data for the 1,800-ft. channel span, notes the character and magnitude of its members, and describes the large concrete and granite piers. It is illustrated by 12 pages of photo-engravings including progress pictures at different stages of the construction of the south anchor and cantilever arms, the steel false-work, the 1,000-ton steel traveler, and the storage yard at the site. The pictures also include special operations of erection, such as the simultaneous hoisting of 160-ton groups of top chord eye-bars for both trusses, and the driving of 12-in. chord pins nearly 400 ft. above water level. The electrical installations from which all power used in erection is provided and the principal machines and tools used for the erection, most of which are special, are also described or illustrated, thus outlining some of the most important points connected with the building of the bridge. (D. R. Kinloch and N. R. McLure, New Liverpool, P. Q., Canada, 35 cents).

A pamphlet full of useful information to the experienced roadbuilder as well as the young engineer has been issued under the title of "The Construction of Macadam Roads," by the Office of Public Roads of the Department of Agriculture. It is written by Mr. A. B. Fletcher, secretary of the Massachusetts Highway Commission, and gives in some detail what is generally considered the best practice in macadam road construction in the Northeastern States, particularly that followed on the Massachusetts roads. It is particularly useful on account of the detailed information it contains respecting the amounts of materials and the average costs of executing different classes of work. The author is careful to point out that direct comparisons of average costs of roads in one locality with those in another are not likely to be of value, for the reason that the conditions in the two localities are rarely even approximately alike. Even the costs of broken stone surfacings, which at first thought might be considered comparable, often lead to wrong conclusions. The data are often interesting, nevertheless, and when used carefully furnish suggestions that are likely to result in some improvements in the methods followed at places where costs run high. The importance of making allowances for different local conditions is shown by the following extract from the pamphlet: "Road officials in New Jersey are advised to use 2,461 tons of broken stone, exclusive of screenings, to the mile of road 15 ft. wide and 6 in. thick, after thorough rolling. The screenings needed are estimated to weigh 407 tons to the mile, making a total weight of 2,868 tons. Trap rock is used almost exclusively in New Jersey. In Massachusetts in 1906 the 6-in. to 4-in. roads, average depth 5 in., built of trap rock and 15 ft. wide, averaged 2,810 tons to the mile, including the screenings. It would appear from these figures that in Massachusetts nearly as much stone is used in building the 5-in. roads as is required in New Jersey for 6-in. work." In addition to the general description of road-building which the pamphlet contains, it has extracts from specifications used in constructing State-aid roads in Massachusetts, New Jersey, New York, Connecticut, Pennsylvania and Maryland, and reproductions of the drawings of standard structures adopted by the Massachusetts Commission.

Letters to the Editor.

BRICK-BATS FOR CONCRETE AGGREGATE.

Sir:—Do you consider concrete made of brick-bats any good for the foundations for a tower of a water-works system, the tower to be of steel and 100 feet high? Do you not think that in time the brick-bats will crumble and allow the tower to settle? City Clerk.

This question is asked so many times that the answer is probably worth printing, as a matter of record. The following notes are taken directly from Mr. L. C. Sabin's "Cement and Concrete," which is the standard American authority on the subject. Fragments of brick and other burnt clay products give good results up to the limit of the strength of the pieces, but this limit is not high. Four beams mixed 1:2:5.8 and tested at the age of eleven months showed an average modulus of rupture of 302 lb., the supports being 4 ft. apart, while two beams of the same composition but made with soft limestone shavings from a stone planer showed an average modulus of 413 lb. Eight beams of brick-bat concrete mixed 1:2:5.8 and tested in 20-in. spans at the age of about two years showed an average modulus of 397 lb., while four beams of the same composition and age, tested under like conditions, made with soft limestone shavings, showed a modulus of 604 lb. It will be seen that the strength obtained with broken brick is considerably lower than that obtained with soft limestone. Had a poorer mortar been used, the brick would, doubtless, have given a better comparative result, since with 1:2 mortar, the brick were not strong enough in themselves to utilize the full adhesive strength of the mortar.

The preceding notes from Mr. Sabin's book state the case so clearly that it is only necessary to emphasize the importance of his assertion concerning the effect of the quality of the brick-bats on the concrete made with them. There is such a great difference in brick that it is out of the question to state in a general way whether brick-bats should be permitted in the foundations of a water tower. Some brick are dense and hard enough to be perfectly safe, provided the unit pressures imposed on the concrete are not excessive, while other brick are unfitted for such use. In this connection it might be stated that a concrete footing for a water tower in a Southern city, which cracked so as to endanger the structure, was made with brick-bats and the cracking was attributed to their use.

REINFORCEMENT STEEL.

Sir:—I note that in your abstract of the paper submitted by Mr. Shuman at the annual meeting of the American Society for Testing Materials, relative to specifications for steel used in the reinforcement of concrete, you neglected any reference to the discussion which was offered by the writer at the time the paper was submitted, and as two out of the three parties whose discussion you quote in your issue of the July 20 are directly interested in the manufacture and sale of the material discussed, you may not consider it unfair to refer to some of the statements made by those not so connected. I am, therefore, enclosing a copy of the discussion which was offered at that time, which, though slightly revised, gives in substance what was stated verbally at the meeting of the American Society for Testing Materials. You are at liberty to use this as you see fit, but I do not think that your paper should favor the tendency which exists, to make it appear that almost any kind of material is good enough for the reinforcement of concrete.

Yours truly,
J. B. French.

Mr. French:—The reading of this paper prompts me to say that as far as my knowledge goes the steel that goes into reinforced concrete is not being tested, and that the purchasers that use it know very little about its properties or quality, and Mr. Shuman being here, he can probably tell us what information he is able to give to purchasers in regard to the physical and chemical properties of the material that is furnished by the company with which he is connected.

It is known that old Bessemer rails are being bought up, twisted hot and furnished to reinforced concrete users, and the people that twist this material cold claim that the hot twisted material is very unreliable, because the phosphorus is excessive, and that it is liable to break and crack in using. Having myself insisted on cold-twisted material, made tests of it and failed to make it bend more than 45° without breaking off short, I have been forced to think that the claims in favor of the cold-twisted material (made from Bessemer stock) may sometimes be very misleading.

If we go back to the manufacturer and ask him for the best information he can furnish us in regard to Bessemer steel that is put into rods for cold-twisting, he gives us what he calls an average analysis. We are not able to find out the extremes between which the results vary, but must be satisfied with the "average." If we try to go a little further and find out how often analyses are made, we are told that an analysis every three or four blows is all that can be furnished.

As far as the statement goes that these average analysis or these analysis of occasional heats or blows represent uniform conditions,— I know, as a matter of fact, from check analyses that have been made within the last two or three months, that we found as high as 13 and 14 points of phosphorus in samples taken from material represented to show less than 10 points, and we have found bars that break before being bent 90°.

I think that in view of the many important uses to which reinforced concrete is now being put, and the failures, many of them involving the loss of human life, which often result from the use of bad material, that steel for the reinforcement of concrete should be as carefully specified, inspected and tested as is considered necessary for structural steel used in bridges and buildings, and if this view is correct, Bessemer steel will be excluded from use in reinforced concrete structures exactly as is already done in the case of important structures built entirely of steel.

Seventy-nine Reinforced Concrete Buildings have been erected or are now under construction in San Francisco, according to Mr. Wm. Ham. Hall. One is a ten-story structure and sixteen are over six stories. Fourteen are stores, thirteen are office buildings, ten are warehouses, six are factories, and two are power stations.